City of Independence Sanitary Sewerage System

Facilities Plan

April 2015

Version 1.0

WATER | ENERGY & RESOURCES | ENVIRONMENT | PROPERTY & BUILDINGS | TRANSPORTATION

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Appendix C Water Quality Trading Memorandum – September 14, 2011.....

Appendix D Lagoon Storage Summary and Water Balance.....

Appendix E Meeting Minutes – November 26, 2008 Independence/Monmouth Planning Workshop.....

Appendix F Reuse Alternatives.....

- *Summary of Combined Reuse Analysis, Monmouth/Independence Joint Effluent Reclaimation Project, Feasibility Report Draft, September 2004 (Whitaker Engineering)*
- *Kennedy/Jenks Technical Memorandum (February 8, 2013)*
- *GHD Review Memorandum (March 22,2013)*

Appendix G Itemized Capital Cost Estimates

ABBREVIATIONS

ADWF	Average Dry Weather Flow
AC	Acres
BOD	Biochemical Oxygen Demand
CDBG	Oregon Community Development Block Grant Program
CIP	Capital Improvement Program
DEQ	Oregon Department of Environmental Quality
DMR's	Discharge Monitoring Reports
DU	Dwelling Units
EDD	Oregon Economic Development Department
FEMA	Federal Emergency Management Agency
Gpad	Gallons Per Acre Per Day
Gpdpc	Gallons Per Capita Per Day
GPM	Gallons Per Minute
HUD	Department of Housing and Urban Development
I/I	Infiltration and Inflow
Lb/day	Pounds per Day
LF	Lineal Feet
LS	Lump Sum
MMDWF	Maximum Monthly Dry Weather Flow
(mgd)	Million Gallons per Day
Mg/l	Milligram per Liter
MMWWF	Maximum Monthly Wet Weather Flow
NPDES	National Pollution Discharge Elimination System
OM&R	Operation Maintenance and Replacement
PDAF	Peak Daily Average Flow
pH	pH Factor
PVC	Polyvinyl Chloride
Q	Flow
Qf	Flow at Full Pipe
Qp	Peak Flow
RCP	Reinforced Concrete Pipe
RM	River Mile
SBR	Sequencing Batch Reactor
SDC	Systems Development Charges
TDH	Total Dynamic Head
TMDL	Total Mass Daily Loading
TSS	Total Suspended Solids
UGB	Urban Growth Boundary
WWPIF	Wet Weather Peak Instantaneous Flow
WWTF	Waste Water Treatment Facility
ZID	Zone of Immediate Dilution

GLOSSARY OF TERMS

Average Dry Weather Flow:	Average of daily flows over the 6-month dry weather period, typically May through October.
Biochemical Oxygen Demand (BOD_5):	The quantity of oxygen utilized in the biochemical oxidation of organic matter under standard laboratory procedure in five days at 20 degrees centigrade expressed in terms in weight and concentration [measurement units are milligrams per liter (mg/l)].
Capital Improvement Program:	A program that contains planned municipal improvement projects over time in accordance with the City's financial plan.
Composite Sample:	A technique where discrete samples are taken at separate times and are combined and treated as one sample.
Effluent:	Treated municipal sewage that flows out of a treatment facility.
E. Coli:	A bacteria found that has to potential to occur in warm bodied animals.
Facultative Lagoons:	A treatment process involving settlement of solids and reduction of organic oxygen demanding material by bacterial activity.
Grab Sample:	A technique where one sample is taken directly and distinctly.
Grit:	Inert solids that settle out a velocities below 1.0 feet per second.
Hypochlorite:	A chemical compound containing chlorine; used for disinfection.
Influent:	Untreated municipal sewage that flows into a treatment facility.
Infiltration and Inflow:	A combination of surface and groundwater that enters into sanitary sewers.
Interceptor:	A mainline backbone of a sewer collection system that typically receives wastewater from a collector sewer or other interceptor.
Maximum Monthly Average Dry Weather Flow:	The maximum monthly average dry weather flow with a 10-percent chance of reoccurrence. Frequently this flow occurs in the month of May.
Maximum Monthly Average Wet Weather Flow:	The maximum monthly average wet weather flow associated with a 20% Probability of Occurrence. In Oregon communities west of the Cascade Mountains, this frequently occurs in January when groundwater is the highest.
NPDES Permit:	A permit that authorizes the discharge of pollution into the US Waterways and is authorized under Section 402 of the Clean Water Act. Permitees must verify permit compliance by monitoring effluent, maintaining records, and filing periodic reports.
Peak Daily Flow:	Peak daily flow resulting from a 5-year, 24-hour storm.
pH:	The standard for measurement of acidity and alkalinity.
Riprap:	A protective layer of large stones used to reduce erosion potential.
Sewer:	A pipe or conduit conveying sewage or wastewater.
Sewage / Wastewater:	A combination of water-carried waste from residences, buildings, and industries in combination with ground and storm water.

GLOSSARY OF TERMS

Spray Irrigation:	Treated and disinfected wastewater used as irrigation for plants and/or crops.
Systems Development Charges:	Fees assessed to new development to reflect the impact on existing infrastructure and future facilities.
Telemetry:	A data stream that reports and transmits information to a separate and remote location.
Suspended Solids:	A measure of the quantity of suspended material contained in the wastewater. The quantity of suspended material present influences the sizing of settling units, sludge handling, and disposal processes, as well as the effectiveness of disinfection.
Urban Growth Boundary:	The boundary around a city where planning and growth is expected to occur. Land outside the UGB will remain rural and land inside the UGB eventually be served by municipal infrastructure.
User:	Any residence or business that contributes sewage or wastewater to the municipal sewer system.
WWTF:	An arrangement of physical, biological and chemical processes used to treat wastewater. The existing treatment facility for the City of Independence is a four cell, controlled discharge, facultative lagoon system with chlorine disinfection.

1 EXECUTIVE SUMMARY

1.1 Introduction

The City of Independence is located in Polk County, Oregon approximately 14 miles southwest of Salem on the west side of the Willamette River. The location of Independence with respect to the State of Oregon is shown in Figure 2.1 located in Section 2 of this report.

This Sanitary Sewerage System Facilities Plan will document the following information:

1. Summary, review and analysis of historic influent flows to the Wastewater Treatment Facility (WWTF).
2. Summary, review and analysis of historic influent and biochemical and suspended solids loading at the WWTF.
3. Historic flow measurements that identify the magnitude of peak flows and a general trend of where they are occurring within the collection system.
4. Identification of WWTF design parameters in conformance with the Oregon Department of Environmental Quality (DEQ) guidelines for Facilities Planning reports.
5. Summary of current and anticipated permit requirements, treatment standards, and monitoring requirements for the City's NPDES permit.
6. Identification of known collection system and WWTF deficiencies and projected future improvements that will be required to accommodate growth and anticipated regulatory changes that have been identified at the time this report was prepared.
7. Identification of alternative treatment technologies and associated capital, labor, training, and operation and maintenance costs necessary to accommodate projected future flows and loads.
8. Completion of a present worth cost effective analysis to compare and assist with identification of a recommended treatment alternative to provide a minimum of 20-years capacity.
9. Identification of a phasing plan to implement recommended improvements in a coordinated and cost effective manner.
10. Identification of the impact on user rates and potential funding sources for the recommended improvements.

In 1998 the City undertook system wide improvements to the collection and treatment facilities. This comprehensive program was successful at dramatically reducing I/I throughout the system, eliminating wet weather overflows, and providing additional

capacity for future growth. Improvements completed as part of the 1998 project went beyond just patching existing problems, and instead, reconfigured the collection and treatment facilities to provide a rational and efficient sewerage system meeting current and projected needs.

Since the completion of the 1998 project, the City has continued to pursue additional capital improvements and program changes to further reduce the impacts of I/I and replace aging sewerage system components. In addition to the capital improvements, the City has aggressively monitored and inspected new construction to ensure that improvements are watertight and in compliance with City and DEQ standards. This forward thinking, planning and program implementation has placed the City in an excellent position to accommodate future development and growth in an efficient, responsible and cost effective manner.

In 2005 the City contracted with David Evans and Associates, Inc. to complete a system wide Wastewater Master Plan that inventoried current conditions, identified existing or anticipated hydraulic or treatment capacity concerns, projected future flows and loads, and identified recommended capital improvements to the collection system and wastewater treatment facility. The Wastewater Master Plan provided the foundation for the City's sanitary sewerage system capital improvement plan, Systems Development Charges (SDC's), and this Facilities Plan. As a key first step to developing an expanded capacity for wastewater management, in 2008 the City contracted with David Evans and Associates, Inc. to complete this ***Sanitary Sewerage System Facilities Plan***. The contract to complete the Facilities Plan was transferred to GHD Inc. when the Project Manager for the engineering consulting team accepted a position with GHD Inc. in March 2010.

1.2 Current Conditions

Population growth since 1998 has resulted in dramatic increases in flows and loads to the WWTF. To date the wastewater treatment facility has demonstrated sufficient treatment capacity to operate in compliance with the current NPDES permit. The City has minimal remaining capacity, and accommodating continued growth will require major capital improvements.

1.2.1 Sewage Collection System

The 1998, a citywide sewerage system improvement project successfully eliminated wet weather overflows within the collection system. Based on recent history and anecdotal reports from City staff, the new pump stations, force mains and interceptors constructed as part of the 1998 project have sufficient capacity to accommodate projected growth tributary to those facilities through the 20-year design period identified in this Facilities Plan.

Some of the perimeter regions of the current City Limits and Urban Growth Boundary (UGB) include land that has adequate sewer service to accommodate existing development, but insufficient capacity to serve the projected needs at full build-out. Since adopting the Wastewater Master Plan and associated capital improvement projects (CIP) in 2006, the City has applied the recommended framework of capital improvements to discussions and conditions of approval for private development. This has guided the extension of sewer mainlines, interceptors and pump stations in a manner that will support organized growth within the UGB. Similarly systems development charges (SDC's) have been updated and assessed to reflect the long term cost and share associated with private development. Future growth will pursue a similar approach that requires private development to participate in a prorated fair share of funding collection and treatment system improvements.

1.2.2 Sewage Lagoon Treatment Facility

The two most immediate concerns are that population growth will produce:

- Summer storage volumes in excess of the available lagoon capacity resulting in recent requests for early discharge to the Willamette River before the permitted discharge period. This has occurred each October during the 2011-2014 study period.
- Organic loads that exceed the biological treatment capacity of the lagoon system.

These constraints will trigger an expansion of the lagoon facility or a change in how the City manages and treats wastewater. Although not as time critical as the hydraulic and biological capacity, additional concerns that will impact the effective treatment of wastewater include:

- Solids accumulation within the lagoon cells has been observed and measured in all four lagoon cells.
- Effluent quality and mass load discharges to the Willamette River are currently within permitted limits but allowed discharges may become increasingly stringent.

- Continued reliance on a facultative lagoon based treatment with a theoretic treatment limit of 30 mg/l for TSS and BOD, will move toward the permitted mass load limits faster than a new and more efficient treatment processes.
- Increased flows and loads due to projected population growth will eventually increase the mass load to the Willamette River in excess of that allowed under the current permit.
- Population growth will eventually push total city population beyond 10,000 which will trigger new and additional monitoring standards.

The current facilities represent a valuable capital resource for the community. During the initial stages of this facilities planning process, the Cities of Independence and Monmouth participated in a workshop to identify potential regional treatment options and order of magnitude cost estimates. The results of that workshop confirmed that completely abandoning the current treatment facility with a new or replacement technology will be far more expensive than constructing additional capacity. At the same time it is recognized that treatment using facultative lagoons will become increasingly unacceptable in terms of land requirements, aesthetics and changing permit requirements. Efficiently expanding wastewater treatment capacity will require the development of alternative treatment option(s), alternative summer flow management, and identifying a reasonable and cost-effective phasing plan for implementation.

1.3 Future Development

Impacts to the WWTF were evaluated using the projected city-wide population growth estimates. Specific land uses and remaining developable lands were used to estimate the impacts within individual sewer basins and the associated major collection system components. However, a detailed hydraulic analysis and model was beyond the scope of this report.

1.3.1 Future DEQ Permit Requirements

Beginning with the preparation of the 2006 Wastewater Master Plan members of the consultant team met with representatives of the Oregon Department of Environmental Quality (DEQ). During the preparation of this Facilities Plan DEQ staff confirmed that the City's NPDES permit remains unchanged since the 2004 renewal. However, DEQ staff noted that the anticipated changes that will be part of the next permit review and renewal cycle will likely include additional monitoring requirements and potentially more stringent pollutant limitations.

Independence and Monmouth share an outfall to the Willamette River. The current NPDES permit allows for discharge during the months of November through May. When the outfall was originally constructed in 1978, the NPDES permit was primarily focused on the mass of total suspended solids (TSS) and biochemical oxygen demand (BOD) present

in the treated effluent. However, since that time additional attention has been given to the impacts of residual chlorine and temperature loads to the receiving stream. Mercury is monitored annually in January for two consecutive sampling events and may be discontinued after that unless otherwise notified by DEQ. Data analysis of the mercury monitoring will be evaluated by DEQ as part of the pending permit renewal and depending on the outcome additional requirements including a mercury minimization plan may be required. Ammonia and dissolved oxygen are monitored monthly during the discharge season. If DEQ determines that there is an impact on the dissolved oxygen concentration in the receiving stream, DEQ may reopen the permit to include an ammonia limit or other conditions or requirements.

The current permits for the Cities are under review by DEQ and although computer models conducted in 1996 indicated that the impacts of discharge were within acceptable limits, continued increase in the magnitude of discharge and the potential for increasingly stringent limits may place new restrictions on the outfall. This could result in the requirement of constructing a multiport diffuser and/or further limiting the number of days when treated effluent can be discharged to the river.

Because the City is estimated to have exhausted the dry weather storage capacity of the existing lagoon system, a modified approach or strategy of managing treated/disinfected effluent must be developed. Four alternative strategies were discussed with DEQ:

- The development of effluent reuse (spray irrigation) facilities for summer months.
- The discharge of cleaner effluent from a mechanical treatment plant during summer months.
- Construction of additional lagoon storage volume.
- Water quality trading in partnership with another NPDES permit holder on the Willamette River.

All four approaches are technically feasible but require further detailed discussions, analysis and approvals by DEQ. It is worth noting that although summer discharge was not ruled out, DEQ staff underscored that in practice obtaining a permit allowing for additional discharge to the Willamette River will be difficult. During the summer months flows in the receiving stream are at their lowest when biological, chlorine toxicity and thermal pollution impacts will be most noticeable. A summer discharge will require a detailed technical and environmental alternatives analysis that clearly demonstrates that the discharge will produce the least impact to the River and the larger ecology and land use.

Water quality trading is a relatively recent option that has been applied on a limited basis in Oregon. Inquiries to DEQ and potential trading partners were made during the preparation of this report. Based on these initial inquiries, there is the potential that water quality

trading could be part of a long term strategy for meeting future NPDES permit requirements. However, at this time it will not solve all of the concerns and challenges associated with population growth. It is most likely that water quality trading will be a valuable strategy for accommodating future permit requirements, or if a partner can be found who is willing to trade an existing summer time discharge and DEQ will allow the transfer of credits to the Monmouth/Independence outfall.

Discussions to date were with industrial facilities that had either discontinued or significantly reduced their operations to an extent where their discharges to the Willamette River are significantly below the permitted levels. Discussion has focused on TSS and BOD mass loading to the receiving stream but future discussions could extend to other permit requirements including but not limited to thermal loads. Potential partners contacted included Weyerhauser, International Paper, Georgia Pacific, and Evanite Fiber Corporation. Appendix C of the Facilities Plan includes a memorandum detailing the discussions and potential options considered.

1.3.2 Population Growth and Projected Flows

It is beneficial to apply population projections during the development of wastewater flow and load estimates. The population history summarized in Table 1.1 demonstrates that Independence has experienced fluctuating periods of rapid and relatively flat growth. Growth from 1980 through 1990 was relatively slow including years with almost no population increase. This corresponded with the decline of timber industry and the discontinuation of several local industrial operations. Shortly after the 1994 Facilities Plan Update, the City began to experience a dramatic increase in population growth including several residential developments. Between 1995 and 2008, the total population increased by 3,155 persons which represented an overall increase of 64%. Annualized this represents an approximate average yearly population increase of 4.0%. The slowdown in the national and local economies between 2008 and 2015 is reflected in a reduced growth rate (0.6% annualized). Recently the City has reported an increase in private development applications and construction, and anticipates a return to population growth. The Portland State University Population Research Center reported the 2013 population as 8,585. The City estimates the 2015 population to be 8,605.

Under estimating or over estimating long term population growth is problematic from a financial point of view and with respect to treatment efficiency. A review of the population history from 1930 through 2015 indicates that during that time period the average population growth rate ranged between 0 and 4%. To strike a balance between the high and low cycles a projected population growth of 2.8% has been used in developing land use planning documents for Polk County, and has been incorporated into the development of this plan. Applying this growth rate to the current population of 8,605 will result in a projected population of 14,949 for the year 2035.

Table 1.1 Historic and Projected Population

Year	Population	Year	Population
1930	1,250	**1985**	4,225
1935	1,325	**1990**	4,425
1940	1,400	**1995**	4,875
1945	1,700	**2000**	6,035
1950	2,000	**2003**	6,850
1955	2,000	**2004**	7,170
1960	2,000	**2015**	8,605
1965	2,250	**2020**	9,879
1970	2,500	**2025**	11,342
1975	3,262	**2030**	13,021
1980	4,024	**2035**	14,949

By analyzing the historic flows and treatment plant loadings, per capita unit parameters for flows and loadings can be established and applied to the projected population growth. Table 1.2 provides a summary of the average values for the ADWF, $WWPIF_5$, TSS and per capita BOD_5 recorded in the 2011-2014 discharge monitoring records.

Table 1.2 Summary of Current System Wide Flows/Loading

Parameter	Units	Value	2011-2014 Unit (per capita)
Average Dry Weather Flow (ADWF)	mgd	0.981	N.A.
Wet Weather Peak Hydraulic Flow ($WWPIF_5$)	mgd	10.7	N.A.
Biological Oxygen Demand (BOD_5)	lb/day	1,814	0.212 lb/day/pc
Total Suspended Solids (TSS)	lb/day	1,671	0.195 lb/day/pc

ADWF = May through October

For reasons described in section 3.2.8 influent flow readings prior to 2011 were noted to be inconsistent. At the time this report was completed influent flow measurements and influent mass loadings including per capita lbs/day loadings from 2011-2014 are believed to be representative of the actual loads delivered to the WWTF.

Table 1.3 lists the parameters utilized for projecting system wide flows and loads. An important underlying assumption used in these projections is that the population, commercial services, and industries that currently live in and serve Independence, will increase proportionally and be of a similar nature in terms the their associated flows and waste strength. If this is not true, and new industries or commercial enterprises with heavy flows and loads develop, these developments will need to be evaluated on a case by case basis to determine if pre-treatment or other methods are appropriate.

Historic average total annual precipitation for Independence is 39-inches. During the monitoring period used in this report (2011 – 2014) the average annual precipitation was

40.9 or approximately at the historic average. However, within this time frame there were years when the annual totals departed from the average significantly. This was particularly noticeable in 2012 when total precipitation was 57.1 inches or approximately 46% greater than the average. In contrast 2013 was significantly dryer with a total of 22.4 inches of precipitation measured at the City's WWTF which is approximately 43% lower than the average annual total.

The parameters noted in Table 1.2 represent historic values and can be used to inform and guide projected future flows and loads. However, the historic per capita contribution to ADWF should be taken in context recognizing that this includes a measureable amount of infiltration and inflow that will not be present in the domestic component of projected future per capita base flows. For this reason a per capita loading of 75 gpcpd is recommended for future estimated increases in base domestic flows as this is consistent with similar cities of this size standards. Future contributions from infiltration and inflow, commercial and industrial users, and diurnal peaking factors will be applied where appropriate.

Table 1.3 Recommended Flows and Pollutant Load Factors

Parameter	
Per Capita Domestic Sewage	75 gpcpd
Infiltration Contribution from New Construction	1000 gpd/acre
Diurnal Peaking Factor	3.0
Existing Inflow Correlation	2.1 mgd/inch of Precipitation
Per Capita BOD Loading	0.212 lbs/day/capita
Per Capita TSS Loading	0.195 lbs/day/capita
Design Storm, 5-Year Recurrence	3.5 inches/day
Average Daily Infiltration	0.20 mgd

The DEQ graphical method was used to estimate current design flows and is contained in Appendix B of the main Facilities Plan document. Using the current design flows, the parameters listed in Table 1.3, and the population projections listed in Table 1.1 future flow and loading values were estimated. Table 1.4 summarizes the existing and projected values.

Table 1.4 Summary of Current and Projected System Wide Flows/Loading

Flow type	Units	2015 value	2035 value
Influent Flow			
Average Dry Weather Flow (ADWF)	mgd	0.981	1.71
Peak Instantaneous Wet Weather Flow (PIF_5)	mgd	10.7	12.4
Influent Load			
Total Suspended Solids (TSS)	lb/day	1,671	2,915
Biochemical Oxygen Demand (BOD_5)	lb/day	1,814	3,169
Total Lagoon Loading Rate (Cells 1, 2, 3 & 4)	lb BOD/acre/day	36	N.A.
Primary Lagoon Loading Rate (Cells 1, 2 and 3)	lb BOD/acre/day	50	N.A.

1.4 Wastewater Treatment Alternatives

Five alternative approaches to wastewater treatment were evaluated. All five alternatives are assumed to address future growth and/or the addition of summer time effluent reuse. From this perspective it has been assumed that at least part of the existing facultative lagoons will remain an integral part of the City's wastewater management strategy during the next 20-years. An additional level of treatment will be provided to address growth and to provide a higher level of effluent treatment and disinfection. This will offer a wider range of opportunities for effluent reuse including spray irrigation for multiple applications.

A second assumption that is inherent to all five alternatives is the need for additional disinfection. The City currently uses chlorine gas which has been an effective and inexpensive approach. The chlorine injection system is nearing the end of its operational life and replacement of key components is assumed. In the coming years it is likely that the issue of chlorine toxicity at the discharge to the Willamette River will become a focus of permit discussions. An updated Mixing Zone Study and model for the outfall shared by the Cities of Independence and Monmouth is currently under review and will help to determine if additional measures are required. With these factors in mind it has been assumed that disinfection using new chlorination/dechlorination facilities will be required under all treatment alternatives considered. If the Mixing Zone Study determines otherwise, dechlorination facilities can accordingly be removed from the recommended capital improvements.

A final assumption common to all treatment approaches is the need to phase construction of new and additional treatment technology. The primary driver for phasing is the capital cost associated with the improvements.

Selecting an appropriate technology for the expanded and replacement wastewater treatment will require a comparative analysis of available processes. Five common and appropriate technologies have been evaluated and are briefly summarized in the following paragraphs:

Alternative No. 1 - Aerated Lagoons:

Under this alternative the existing facultative lagoon system would be retrofitted to add mechanical aeration. Aeration of lagoons can be accomplished using a wide variety of proprietary processes that fall into general categories as either floating or diffused aerators. The addition of aerators increases the amount of oxygen that is made available to the organisms in the lagoons allowing for higher loadings per acre of lagoon. Aerated lagoons require electrical power service to drive the aerators, some structural modifications to the lagoons, and means of managing the on-going production of solids that are inherent to the aeration process.

Alternative No. 2 - Activated Sludge:

Activated sludge is characterized by a series of physical and biological processes. After primary treatment consisting of grit removal and settling, air is bubbled through the wastewater creating an environment that encourages biological reduction of the waste. The system is optimized by adding recirculated organisms to develop a biological floc that reduces the organic content of the sewage. The treatment process separates the solids (sludge) and liquid (supernatant) then provides additional treatment and disinfection. At the completion of treatment, disinfected effluent is discharged to the receiving stream or spray irrigated. Solids are further concentrated for disposal either in a landfill or through land application.

Alternative No. 3 - Oxidation Ditch:

Oxidation ditches consist of large round or oval tanks fitted with disc or brush aerators that circulate wastewater around the ditch as well as provide aeration. Similar to activated sludge systems additional clarification tanks and settling tanks are required to address the separated liquid and solids.

Alternative No. 4 - Sequencing Batch Reactor (SBR):

The SBR process generally consists of two identically configured and equipped tanks that have a flow through system of raw wastewater coming in and treated effluent being discharged. The tanks alternate cycles with one tank filling and aerating while the second tank is being allowed to settle and decant. Similar to the activated sludge process the system is optimized by recirculating organisms to seed the process. Also similar to other mechanical processes additional tanks are required to address the final treatment and disinfection of the separated liquids and solids.

Alternative No. 5 - Membrane Bioreactor (MBR):

The MBR process uses a combination of a bioreactor and a membrane filter. The bioreactor process is similar to other treatment technologies that recirculate organisms to seed the biological process. Treated wastewater is then passed

through membranes that retain solids and microorganisms. MBR systems fall into two general categories: immersed and external/sidestream membranes. Solids and liquid management and disinfection systems are required. Although it requires additional chemical processes, MBR's can be configured to produce effluent with very low levels of nitrogen and phosphorus.

A detailed description and comparison of advantages and disadvantages associated with each alternative is provided in Chapters 5 and 6 of the Facilities Plan.

1.5 Treated Effluent Spray Irrigation Alternatives

Management of treated effluent is particularly critical during the summer months when relatively low flows in the Willamette River prohibit discharge. The City currently stores flows during the months of June through October. This approach will become increasingly infeasible as projected population growth will require extremely large areas of land to be dedicated for additional lagoon cells. A more practical approach with additional side benefits is the development of summer effluent reuse of which spray irrigation is one of the most common.

Treated and disinfected effluent can be recycled and applied as irrigation to crops and open spaces. The range and type of areas where this is allowed is dependent on the level of treatment and disinfection provided. Increasing the level of treatment through one of the five treatment technologies previously described, will allow for application on a wide range of land uses including crops, open spaces, public parks and landscaping.

The City of Monmouth has successfully developed spray irrigation for agricultural land south of the Monmouth city limits. During the course of preparing this Facilities Plan several inquiries and evaluations were completed to determine if becoming a "customer" to Monmouth and discharging treated effluent at their spray irrigation sites was feasible. After several attempts it was deemed to be infeasible due to several factors including:

1. The cost to "buy in" allowing for reimbursement of Monmouth's sunk cost far exceeded the cost to develop an independent facility.
2. The on-going energy consumption and associated cost required to pump effluent from the Independence lagoons to the Monmouth facility (approximately 3 miles) was inefficient and represented a potentially volatile operational cost.

A preferred recycled effluent option is for Independence to develop separate facilities at a location closer to the current and proposed WWTF. Several agricultural property owners and businesses near the Independence UGB were approached and the City was able to successfully enter into preliminary negotiations for multiples sites with sufficient aggregate area providing an estimated capacity sufficient to serve beyond the next 20 years. A "Preliminary Letter of Interest" was signed by the City and Lafayette Farms in October 2013.

1.6 Spray Irrigation Alternatives

Spray irrigation is regulated by *OAR 340-055 Recycled Water Use*. Although the details are provided in the OAR's, some critical elements are summarized in the following tables:

Table 1.5 Spray Irrigation Requirements

Spray Irrigation Treatment Requirements					
	Non-Disinfected	**Class D**	**Class C**	**Class B**	**Class A**
Biological Treatment	X	X	X	X	X
Disinfection		X	X	X	X
Clarification				X	X
Coagulation				X	X
Filtration					X
Public Access And Buffers					
	Non-Disinfected	**Class D**	**Class C**	**Class B**	**Class A**
Public Access	Prevented	Controlled	No Contact During Irrigation	No Contact During Irrigation	None
Buffers, Sprinkler	Site Specific Setbacks	Surface 10/'100', Spray Site Specific	10'/70'	10'	
Allowable Uses					
	Non-Disinfected	**Class D**	**Class C**	**Class B**	**Class A**
Agricultural	Few	Few	Most	Most	All
Urban/Irrigation	No	No	Some	Some	Yes
Commercial/Industrial	No	No	Yes	Yes	Yes
Construction	No	No	Yes	Yes	Yes
Impoundment	No	No	Some	Some	Yes

The selection of appropriate spray irrigation sites is dependent on a number of factors including:

- Soil type
- Crop type
- Level of Treatment and disinfection
- Potential ground water impacts
- Proximity to surface water and potable water sources.
- Intended land use
- Characteristics of the wastewater (phosphorus, nitrogen)
- Time of year for irrigation
- Growing season
- Climate

Irrigation for agricultural crops is dependent on the type of crop and growing season. Because the application rate and frequency is critical to the successful cultivation of a marketable crop it is important that an adequate volume of storage is available to buffer the continuous production of treated effluent from the periodic irrigation demand. The following is a summary of typical application rates and land requirements associated with crops in the Willamette Valley. These values should be considered general in nature and final selection of a site and sizing of associated irrigation components should be selected based on refined irrigation and hydraulic calculations, preliminary design report, and a *Recycled Water Use Plan* in compliance with OAR 340-055.

Table 1.6 Land Requirements for Typical Crops

			Land Requirements			
	Net Irrigation in/yr	**Net Irrigation gal/acre/yr**	**2025 Excess Volume Stored mg**	**2025 Area Required acres**	**2035 Excess Volume Stored mg**	**2035 Area Required acres**
Alfalfa	16.3	442,584	75	169	118	266
Filberts	22.0	597,353	75	125	118	197
Grapes	10.1	274,239	75	273	118	430
Grass Seed	18.6	505,035	75	148	118	234
Pasture	18.4	499,604	75	150	118	236
Pears	19.8	537,618	75	140	118	219
Poplar Trees	42.0	1,140,401	75	66	118	103

The values noted for “Volume Stored” in Table 1.6 represent total volumes including both existing and projected ADWF assuming an average year precipitation.

The primary purpose of developing a spray irrigation program to dispose of treated and disinfected effluent is to provide for the disposal of effluent during the dry weather holding period. The current NPDES permit requires that the City discontinue discharges to the Willamette River from the period of June 1st through October 31st (153 calendar days).

During this period all influent is stored in the facultative lagoons. As noted previously as a result of population growth and expanded commercial and industrial interests, the City is approaching the limits of the available storage volume. At a minimum, recycled effluent spray irrigation facilities will need to be developed to accommodate flows associated with future growth. In practice, development of spray irrigation facilities should allow for an anticipated transition to spray irrigation of the majority of summer flows, current and projected by the year 2035. Table 1.7 provides a summary of current and projected flows and loads to the WWTF.

Table 1.7 Projected Flows and Loads

Year	Population	ADWF (mgd)	$PADF_5$ (mgd)	PIF_5 (mgd)	$MMDWF_{10}$ (mgd)	$MMWWF_5$ (mgd)	BOD_5 (lbs/day)
		Spray Irrigation, Summer Storage	Headworks, all process basins	Influent Pump Stations, Flumes	Tertiary Treatment	Biological Treatment, Solids Processing	Biological Treatment, Solids Processing
2015	8,605	0.981	8.30	10.7	1.42	2.21	1,814
2020	9,879	1.13	8.45	11.0	1.57	2.36	2,094
2025	11,342	1.30	8.61	11.4	1.73	2.52	2,405
2030	13,021	1.49	8.81	11.9	1.93	2.72	2,760
2035	14,949	1.71	9.03	12.4	2.15	2.94	3,169

Outfall = 2.0 mgd (2009 Permit, 30mg/l max monthly average, 500 lbs/day)

1.7 Cost Comparison of Capital Improvement Alternatives

Managing wastewater during the coming years will require a combination of new treatment processes and disposal or reuse technologies. Some are more adaptable to anticipated regulatory changes and public expectations than the current approach which relies on facultative lagoons, wet weather discharge to the Willamette River, and summer month storage.

As a general rule the more adaptable and flexible a system of treatment and reuse, the more expensive it is to operate and maintain. However, in general these types of costs are relatively small and the long term environmental and cost benefits outweigh the differences in annual operation and maintenance. Capital costs, O&M and combined present worth costs are summarized in the following table. The additional benefit of using proven yet more “state of the art” treatment technologies is the increasing efficiencies. MBR treatment in particular is an industry focus that has achieved dramatic increases in efficiencies during the last decade with consequent reductions in area “foot prints” and lower operating and maintenance costs. The technology is far enough past initial development and application to have a proven track record of reliable service, but still new enough that significant decreases in capital cost are anticipated in the coming decade.

One benefit to the City is that it is likely future phases adding additional treatment capacity using MBR technology will be increasingly cost effective.

Development of a higher level of treatment will also provide the potential opportunity for water quality trading with the City of Monmouth or other NPDES permit holders discharging to the Willamette River. Lower levels of BOD, TSS, nitrogen and phosphorus that can be achieved using the MBR process are many of the same pollutants that have been identified as potentially more restrictive under the future permit renewals. The amount of reduction of these pollutants using modern technology is significant and in some cases potentially an order of magnitude (30 mg/l to 3 mg/l). It is conceivable that the City would be able to meet its own future permit requirements, and have sufficient treatment improvements and capacity to sell credits to another discharge partner.

Table 1.8 Summary of Estimated Capital Improvement Programs

	PROJECT	WASTEWATER TREATMENT IMPROVEMENTS	IRRIGATION / REUSE IMPROVEMENTS	TOTAL
CAPITAL COST - TREATMENTAND REUSE IMPROVEMENTS				
T-1	ALTERNATIVE NO. 1 -LAGOON AERATION RETROFIT	$10,409,553.00	$3,244,472	$13,654,025
T-2	ALTERNATIVE NO. 2 - ACTIVATED SLUDGE	$13,869,410.94	$3,244,472	$17,113,883
T-3	ALTERNATIVE NO. 3 - OXIDATION DITCH	$13,023,125.50	$3,244,472	$16,267,598
T-4	ALTERNATIVE NO. 4 - SEQUENCING BATCH REACTOR (SBR)	$12,240,613.00	$3,244,472	$15,485,085
T-5	ALTERNATIVE NO. 5 - MEMBRANE BIOREACTOR (MBR)	$11,091,927.00	$3,244,472	$14,336,399
ANNUAL OPERATING COST - TREATMENTAND REUSE IMPROVEMENTS				
O-1	ALTERNATIVE NO. 1 -LAGOON AERATION RETROFIT	$476,620	$28,405	$505,025.00
O-2	ALTERNATIVE NO. 2 - ACTIVATED SLUDGE	$471,775	$28,405	$500,180.00
O-3	ALTERNATIVE NO. 3 - OXIDATION DITCH	$479,635	$28,405	$508,040.00
O-4	ALTERNATIVE NO. 4 - SEQUENCING BATCH REACTOR (SBR)	$446,770	$28,405	$475,175.00
O-5	ALTERNATIVE NO. 5 - MEMBRANE BIOREACTOR (MBR)	$464,360	$28,405	$492,765.00
PRESENT WORTH COST - TREATMENTAND REUSE IMPROVEMENTS				
PW-1	ALTERNATIVE NO. 1 -LAGOON AERATION RETROFIT	$16,886,819	$3,630,496	$20,517,315
PW-2	ALTERNATIVE NO. 2 - ACTIVATED SLUDGE	$20,280,833	$3,630,496	$23,911,329
PW-3	ALTERNATIVE NO. 3 - OXIDATION DITCH	$19,541,365	$3,630,496	$23,171,861
PW-4	ALTERNATIVE NO. 4 - SEQUENCING BATCH REACTOR (SBR)	$18,312,217	$3,630,496	$21,942,713
PW-5	ALTERNATIVE NO. 5 - MEMBRANE BIOREACTOR (MBR)	$17,402,579	$3,630,496	$21,033,075

(1) Irrigation/Reuse Improvements assume irrigation at Lafayette Farms.

(2) Present Worth assumes 20-year period with O&M, R Costs at an annual discount rate of 4%

1.8 Recommended Improvements and Phasing Plan

Chapters 5 and 6 identify several treatment technologies that are appropriate and can provide excellent treatment and management of wastewater during the next 20 years. Preliminary budget cost estimates for capital improvements and annual operation and maintenance are developed to allow for an initial financial comparison. Additional operational and permit considerations are identified along with qualitative comparison of issues and concerns typically associated with wastewater management.

In addition to the treatment of wastewater, an efficient and comprehensive method of managing effluent is required. Alternatives for the management and/or reuse of treated and disinfected effluent are identified in Chapter 5 and associated budget level cost estimates developed in Chapter 6.

Identifying the most cost effective, efficient and appropriate approach to managing wastewater during the next 20-years and beyond requires evaluating a systems approach that considers both treatment and effluent management. A recommended system wide approach is detailed at the end of Chapter 6 which will serve the City well.

As part of this capital improvement program the City will develop a spray irrigation effluent reuse program (*Recycled Water Use Plan*). The spray irrigation program will become an integral part of the City's sewerage system and land use development policy. For this reason a detailed process of executing legal agreements, and development and approval of a *Recycled Water Use Plan (RWUP)* by DEQ must be completed. The improvements should be constructed in Phases. A renewal of the City's NPDES permit will be required to obtain approval of a RWUP.

1.8.1 2017 Capital Improvements – Phase I

Development of effluent reuse facilities to allow for recycling of treated and disinfected wastewater through spray irrigation on agricultural properties near the City's UGB. This will address the most immediate capacity concern with wastewater management, the storage of dry weather flows.

1.8.2 2020 Capital improvements – Phase II

The second phase will address the need for additional biological treatment associated with population growth through the year 2030. Capital improvements associated with this second phase are recommended to begin construction during the year 2020. The total $MMWWF_5$ is expected to increase by approximately 0.75 mgd (2.21 mgd to 2.94 mgd) by 2035. Assuming straight line growth Phase I treatment would require construction of a treatment facility capable of providing 0.50 mgd.

1.8.3 2030 Capital improvements – Phase III

The third phase will focus on the development of additional treatment and effluent reuse capacity to accommodate growth anticipated between the years 2030 and 2035. Construction of capital improvements associated with this third phase are anticipated to be required and begin during the year 2029. An additional treatment plant capacity of 0.25 mgd will be constructed as part of this phase.

1.9 Phased – Capital Cost Estimate

The estimated capital cost for construction of the recommended improvements is summarized in the following sections. The estimated cost includes the cost of construction and equipment, and an estimated budget for "soft costs" including land acquisition, survey, design, legal, permitting, and construction management. As a budget level cost estimate a 30% project contingency is included. This contingency will be reduced as the definition of the project is further refined and sizing of specific components is better developed.

1.9.1 2017 Capital improvements – Phase I

Phase I capital cost associated with constructing an effluent recycling/land application system to accommodate summer storage requirements are summarized in Table 1.9.

Table 1.9 Phase I – Recycled Effluent (Spray Irrigation)

Summary Phase I Improvements		
Item	**Description**	**Cost**
1	**Irrigation Pump Station**	
a.	Pumps at Independence WWTF	$100,000
b.	Structural Improvements	$55,000
c.	Mechanical Improvements	$45,000
d.	Electrical Improvements	$55,000
e.	Strainer and Backwash System	$75,000
2	**Pressure Main and Connection Points**	
a.	12" Force Main	$976,800
b.	User Connection Points	$120,000
c.	Irrigation Set	$630,000
d.	Easements	$50,000
	Total Construction Cost	**$2,106,800**
	Soft Costs and Contingency @ 54%	**$1,137,672**
	Total Cost	**$3,244,472**

1.9.2 2020 Capital improvements – Phase II

Phase II capital costs associated with constructing MBR treatment facility to accommodate projected future biological treatment requirements are summarized in Table 1.10.

Table 1.10 Phase II Recommended Alternative - Membrane Bioreactor (MBR)

Summary Phase II Improvements		
Item	**Description**	**Cost**
1	Headworks	$458,200
2	Process	$2,461,800
3	Operation and Control	$493,000
4	Effluent Management	$390,200
5	Piping Reconfiguration	$44,800
6	Solids Management	$600,000
	Total Construction Cost	**$4,448,000**
	Soft Costs and Contingency @ 54%	**$2,401,920**
	Total Cost	**$6,849,920**

1.9.3 2030 Capital improvements – Phase III

Phase III will involve expanding the treatment capacity of the MBR system to accommodate flows associated with projected growth between the years 2030 and 2035. Relative to Phases I and II the capital costs associated with Phase III will be lower on a "per gallon treated" basis. This is because major improvements completed during he first two phases will include capacity of key components (headworks, chlorination/dechlorination, solids management, effluent reuse) sufficient to accommodate this time period. Phase III will consist of adding additional MBR treatment modules or trains in parallel to those constructed as part of Phase II. The total capital cost associated with Phase III improvements is estimated to be **$4,242,007** as summarized in Table 1.11.

Table 1.11 Phase III Recommended Alternative - Membrane Bioreactor (MBR) Expansion

Summary Phase III Improvements		
Item	**Description**	**Cost**
1	Headworks	$67,700
2	Process	$1,902,500
3	Operation and Control	$73,900
4	Effluent Management	$89,500
5	Piping Reconfiguration	$3,450
6	Solids Management	$617,500
	Total Construction Cost	**$2,754,550**
	Soft Costs and Contingency @ 54%	**$1,487,457**
	Total Cost	**$4,242,007**

2 INTRODUCTION

2.1 Authorization

In August 2008 David Evans and Associates, Inc. and the City of Independence entered into a formal contract to prepare this Sanitary Sewerage System Facilities Plan. In April 2010 the project manager for the consulting team joined GHD, Inc. and the contract was transferred to GHD, Inc. for completion.

2.2 Purpose and Scope

This 2015 ***Sanitary Sewerage System Facilities Plan*** is developed with attention to the following goals:

1. **Accommodate Growth:** Provide treatment and collection system capacity to accommodate growth through the year 2035 in an orderly, environmentally responsible and cost-effective manner.
2. **Meet Requirements of NPDES Permit**: Comply with the stated requirements of the City's discharge permit with DEQ, and wherever possible, anticipate the impacts of future modifications. Provide 80-percent BOD_5 and 65-percent TSS removal. Contain and treat all of the sewage flows resulting from a 5-year, 24-hour storm. Have the flexibility to upgrade the treatment process to remove nitrogen and other pollutants.
3. **Enhance Open Space Habitat**: Provide future recreational open space and habitat enhancement, where feasible and economical.
4. **Conserve Water**: If deemed economically feasible and justifiable, provide the ability to re-use treated effluent including spray irrigation in lieu of discharge to the Willamette River.
5. **Minimize Cost to Rate-Payers**: Construct the most cost-effective facility that meets regulatory, environmental and resource protection requirements and goals. Use the existing wastewater facility to the greatest extent possible. Provide automated, operator-friendly features applying appropriate technology.
6. **Be A Good Neighbor**: Provide facilities and improvements that are compatible with properties and uses adjacent to the wastewater treatment facilities and properties.
7. **Public Safety**: Ensure safety and security requirements.

2.3 Study Area

The study area for this report encompasses all of the area contained within the Urban Growth Boundary (UGB). The study area includes two significant waterways. The Willamette River establishes the eastern boundary of the study area, and Ash Creek flows through the center splitting into the North and South Forks at approximately the center of the City Limits and UGB. The portions of the Willamette River and Ash Creek that are contained within the study area include 100-year floodways and flood plains defined by FEMA. Many of the existing and proposed sewerage facilities are located within or immediately adjacent to the floodway/flood plains.

Figure 2.1 located on the following page shows the location of Independence with respect to the State of Oregon. Figure 2.2 includes a map of the overall Study Area (UBG) and the existing service areas (City Limits).

2.4 Planning Process

The consultant engineering team met and spoke regularly with City staff and the Oregon Department of Environmental Quality (DEQ) during the drafting, review and preparation of the Facilities Plan. Meetings with City staff focused on obtaining and verifying the most up-to-date information, assessing operator's needs and preferences, and examining the existing facilities. Communication with representatives of the Oregon DEQ included discussions regarding appropriate technology, current and anticipated regulatory requirements and permit regulations, and potential water quality trading opportunities that could impact the City.

During the early stages of the Facilities Planning process city staff from Independence and Monmouth, along with engineering consultants for each city, met to discuss options for combining treatment and disposal facilities. This work included several face to face meetings and an all day workshop. Alternatives were identified and evaluated in terms of technical merit, preliminary cost estimates, and impacts to user fees.

The initial Draft Facilities Plan (2009) and back ground information was provided for review by the Independence City Council at two preliminary stages. Formal public input was solicited during a Public Open House on July 8, 2009. On July 28, 2009 the Draft Facilities Plan was presented to the City Council for consideration and review. The *Final Sanitary Sewer Facilities Plan* will be submitted for approval by City staff and the City Council.

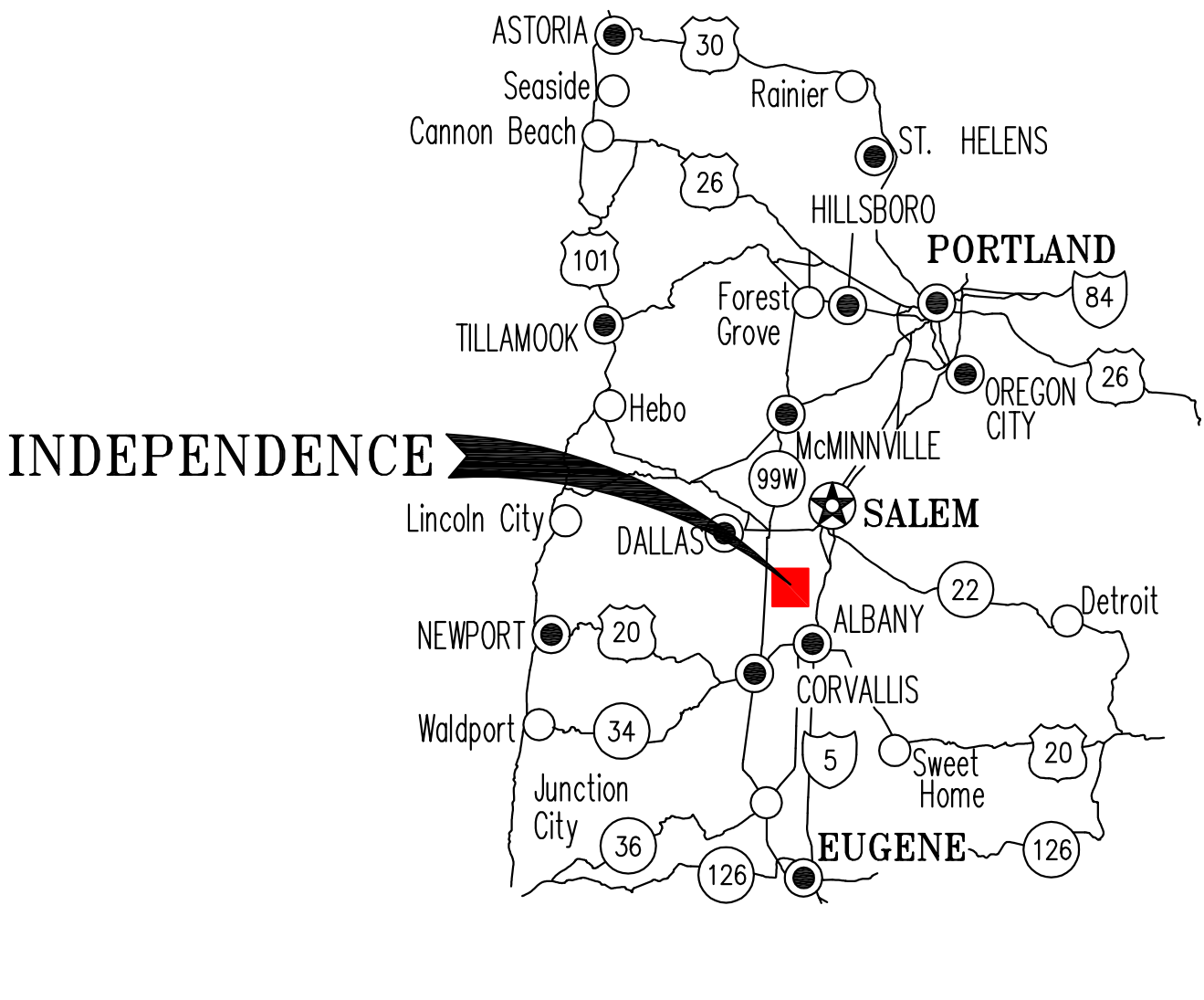
At the time the Draft Facilities Plan was under consideration, economic changes were beginning to impact the region affecting population growth and financing opportunities for municipalities. In response to national and local changes to the economy, the completion of the Facilities Plan was put on hold while the City staff and consulting team investigated other options to meeting the immediate and long terms needs of wastewater management.

This included:

- Consideration of potential water quality trading with other permitted discharges to the Willamette River.
- Consideration of interim treatment and disposal measures that while not as technically efficient would reduce the initial capital expenditure for expanding wastewater facilities.
- Developing a phasing plan to allow for implementing the required capital improvements in steps that are responsive to growth but allow the large capital expenditures to be incurred in a step wise manner at 2 or 15 year increments.

This investigation of alternative approaches included meetings and discussions with representatives of DEQ, funding agencies, and private entities with permitted discharges to the Willamette River. It was also closely coordinated with the financial consultant working with City staff to address the long term financial health for the City.

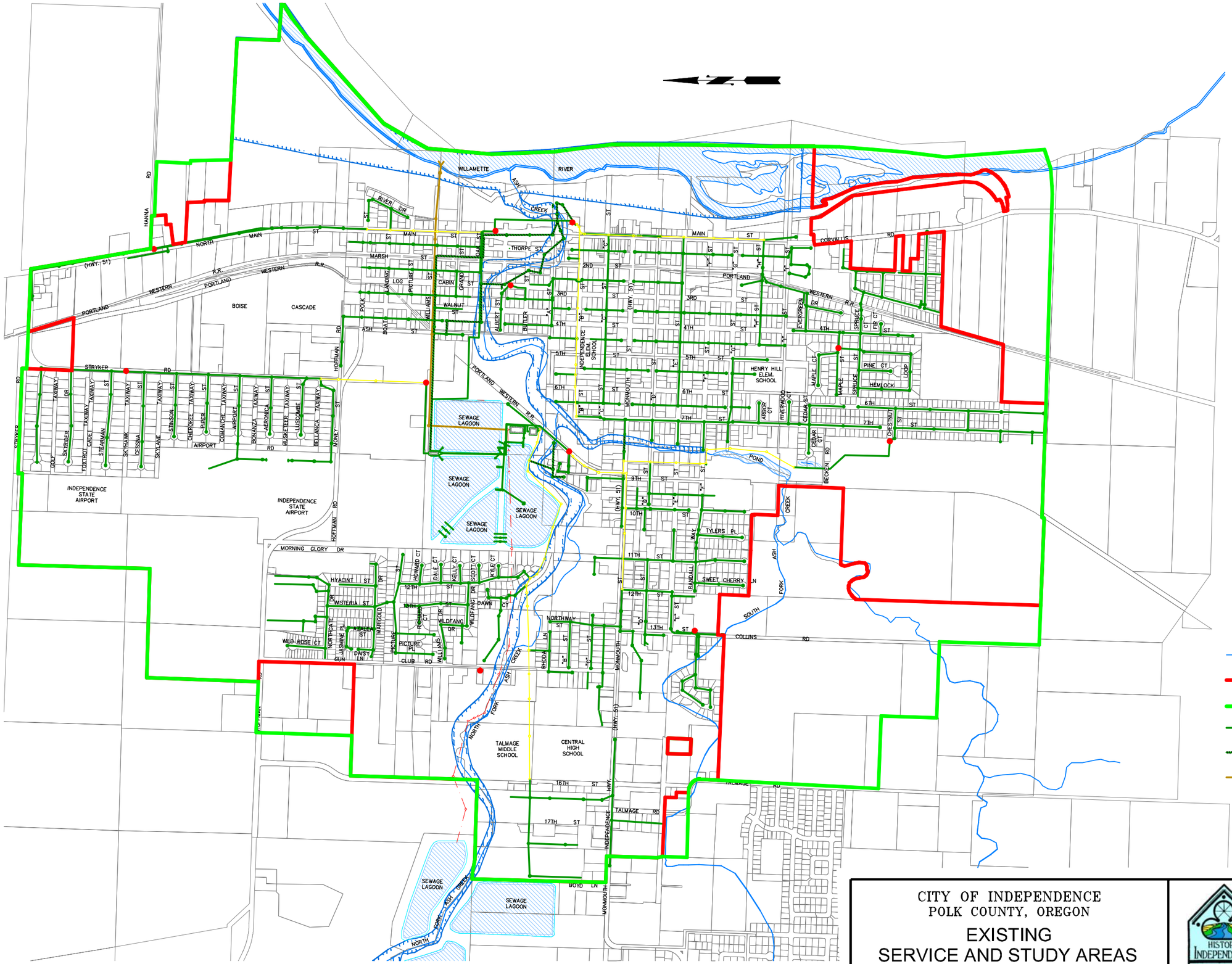
Several private NPDES permit holders were contacted to inquire about their interest in forming a partnership for water quality trading. Although there was some interest in the concept, at the time this plan was completed no draft agreement was pending.



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

VICINITY MAP

FIGURE 2.1



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

EXISTING
SERVICE AND STUDY AREAS

HISTORIC INDEPENDENCE

FIGURE 2.2

2.5 Existing Sanitary Sewer Planning Documents

The following documents represent previous planning efforts for the City sanitary sewerage system and were reviewed in the preparation of this Facilities Plan:

- "Independence Wastewater Masterplan", *Version 1.0, October 10, 2005*
- "Sanitary Sewer Preliminary Design Report", *City of Independence, Oregon, 1996*
- "The City of Independence Sewerage Facilities Plan Update, Volume I and II – Report", *August 1994*
- "Television Inspection Reports of the City of Independence", *Insituform Technologies, 1996*
- "Smoke Testing Reports of the City of Independence", *Insituform Technologies, 1996*

In addition to these recent reports, C&G Engineering, Inc. prepared a 1977 Sewer System Evaluation Survey (SSES) for the Cities of Monmouth and Independence. The SSES report was provided for GHD's review. Although much of the information in these reports is outdated, some of it still describes condition of portions of the existing collection system, and provides some history for the City's sewerage system.

3 CURRENT CONDITIONS

The earliest portions of the Independence collection system were constructed in 1925. In the early 1950's the Cities of Monmouth and Independence constructed a two-city primary treatment plant. The collection system for Independence was originally configured to direct all wastewater generated within the City's service area to the primary treatment plant at the confluence of Ash Creek and the Willamette River. This system was subject to flooding by the Willamette River and Ash Creek, and in the 1960's became overloaded as a result of population growth and excessively high wet weather related flows.

In 1963, the two cities constructed separate lagoon treatment facilities and a common effluent outfall to the Willamette River. As part of the lagoon construction project, the original treatment plant was converted to a lift station (Original Riverview Pump Station) to pump the City's wastewater west to the newly constructed lagoon treatment facility.

In 1978 Independence expanded the lagoon treatment through the construction of two additional lagoon cells. Monmouth undertook a similar expansion and the two communities shared the construction of a new 36-inch diameter outfall to the Willamette River.

In 1998 Independence completed system wide improvements to the collection and treatment facilities. The modifications to the collection system resulted in a reconfiguration of the sewerage basins resulting in a more rational approach that improved hydraulic efficiency of the overall system, and lowered the hydraulic gradeline of critical sections. Prior to the construction of these improvements the collection and treatment systems were overloaded during wet weather resulting in frequent overflows. The 1998 improvements included:

Riverview Pump Station: Demolition of the original treatment facility (also original Riverview Pump Station) and replacement with a new submersible station and new 16-inch force main to the WWTF.

I/I Remediation - North Interceptor: Elimination of approximately 1,000 feet of the North Interceptor located within the floodplain that was characterized by heavy root intrusion and visual holes and cracks.

I/I Remediation - Creek Interceptor: Elimination of the Creek Interceptor that ran parallel to Ash Creek and was located within the flood plain and below the 100-year flood elevation with visible holes and corroded rot.

New Pump Stations and Force Mains: In addition to the replacement of the Riverview Pump Station, three additional new pump stations were built to allow for the reconfiguration of the collection system resulting in an increase in hydraulic capacity, and eliminating the need for the segments of the North and Creek interceptors located within the flood plain. The three additional new pump stations

included 9th Street, Albert Street, and Oak Street Pump Stations. Associated forcemains were constructed to convey flow directly or indirectly to the WWTF.

I/I Remediation – Middle Interceptor: Replacement of approximately 1,500 feet of the Middle Interceptor with larger diameter pipe and a deeper pipe alignment. All Middle Interceptor flows west of Ash Creek were redirected into the new 9th Street Pump Station.

Telemetry: Installation of full reporting telemetry at each of the new or replacement pump stations, and at the Stryker Road, Maple Street, Hanna Road (North), and 13th Street Pump Stations.

Headworks Improvements: Construction of a new influent vault and 12-inch Parshall flume at the WWTF to allow for the centralized measurement of wastewater influent. Installation of influent measurement and recording equipment, and a composite sampler at the influent Parshall flume.

Lagoon Transfer Structure: Construction of a new transfer structure and piping between Lagoon Cell Nos. 2 and 3 allowing for more efficient decanting of the partially treated effluent.

Disconnection of Roof Drains: Disconnection of roof drains and storm sewer catch basins identified during the 1996 smoke testing.

Sanitary Sewer Laterals: Replacement of sanitary sewer service laterals previously identified as leaking during smoke testing in 1996.

The 1998 improvements dramatically reduced I/I and wet weather flows to the treatment facility, and eliminated overflows within the collection system. Prior to 1998 influent to the treatment facility was not directly measured so an accurate and precise determination of the overall percent of I/I removed was not possible. However, the I/I reduction achieved by the 1998 improvements is considered sufficient to allow the City to convey and treat the remaining wet weather sewage flows as no overflows have been reported or noted. Since the 1998 project was completed, the City has further reduced I/I by implementing additional measures recommended in the 1996 Preliminary Design Report. In addition to city funded capital improvement projects the City has aggressively monitored new construction to ensure that new sewer and structures are water tight.

Several exposed, severely corroded, open and cracked pipes were present in alignments adjacent to Ash Creek and in the flood plain. It was clear that prior to 1998 during much of the winter infiltration entering the system from these sources was limited only by the hydraulic capacity of the associated collection pipes and downstream pump station. Reconfiguring the overall collection system allowed for removal of these interceptors. A very rough estimate using the hydraulic capacity of the deficient pipes indicates that these inflow sources could have contributed flows in the order of 10 mgd. This compares to the estimated 10.7 mgd peak instantaneous flows currently associated with a 5-year storm.

The current sanitary sewerage system serving the City of Independence consists of a gravity and pumped collection system, and a lagoon based wastewater treatment facility (WWTF). Treated effluent is measured, chlorinated then discharged to a 36-inch diameter outfall line which flows east approximately 3/4 of a mile before discharging into the Willamette River. The outfall line is shared with the City of Monmouth.

In general, the WWTF has performed adequately and operated within the permitted tolerances required for effluent concentrations and mass loadings. However, although the general pattern of performance for effluent concentrations of BOD_5 and TSS, and total mass loads has remained below the permitted tolerances, the general trend has been upward reflecting the increase in population. Due to the buffering characteristic of a lagoon system, peak hydraulic and organic loadings will tend to even out, and to some extent mask a larger trend toward reaching design capacity. It is important to recognize and anticipate that flows and loads are increasing and that performance will continue to degrade at a relatively gradual rate until design capacity is reached. As the influent concentrations and flows reach the design capacity, it is likely that a dramatic change in performance will occur and be evident through abrupt increases in effluent concentrations. Additional treatment capacity, and/or modifications to the current treatment process are necessary and should be anticipated in the immediate future.

Due to a period of poor treatment quality during the fall of 2010 the City received notice and was fined by DEQ. In response, City staff made several modifications to standard operating procedures and have operated the lagoon system within the permitted limits since that action. A detailed analysis and discussion is provided in Chapters 4 and 5 of this Facilities Plan.

3.1 Existing Sewage Collection System

The existing collection system consists of five major basins, labeled alphabetically A through E. Within the five basins wastewater flows primarily by gravity to one of five major pump stations that convey the wastewater to the treatment facility (WWTF) through force mains (pressurized sewers). The collection system pipe materials include vitrified clay, concrete and PVC. Although all sewers constructed since the 1980's are polyvinyl chloride (PVC), many of the segments using older clay or concrete materials are "leaky" and contribute much of the City's I/I. The newer PVC pipe is considered to be relatively watertight.

3.1.1 Existing Collection System Configuration and Components

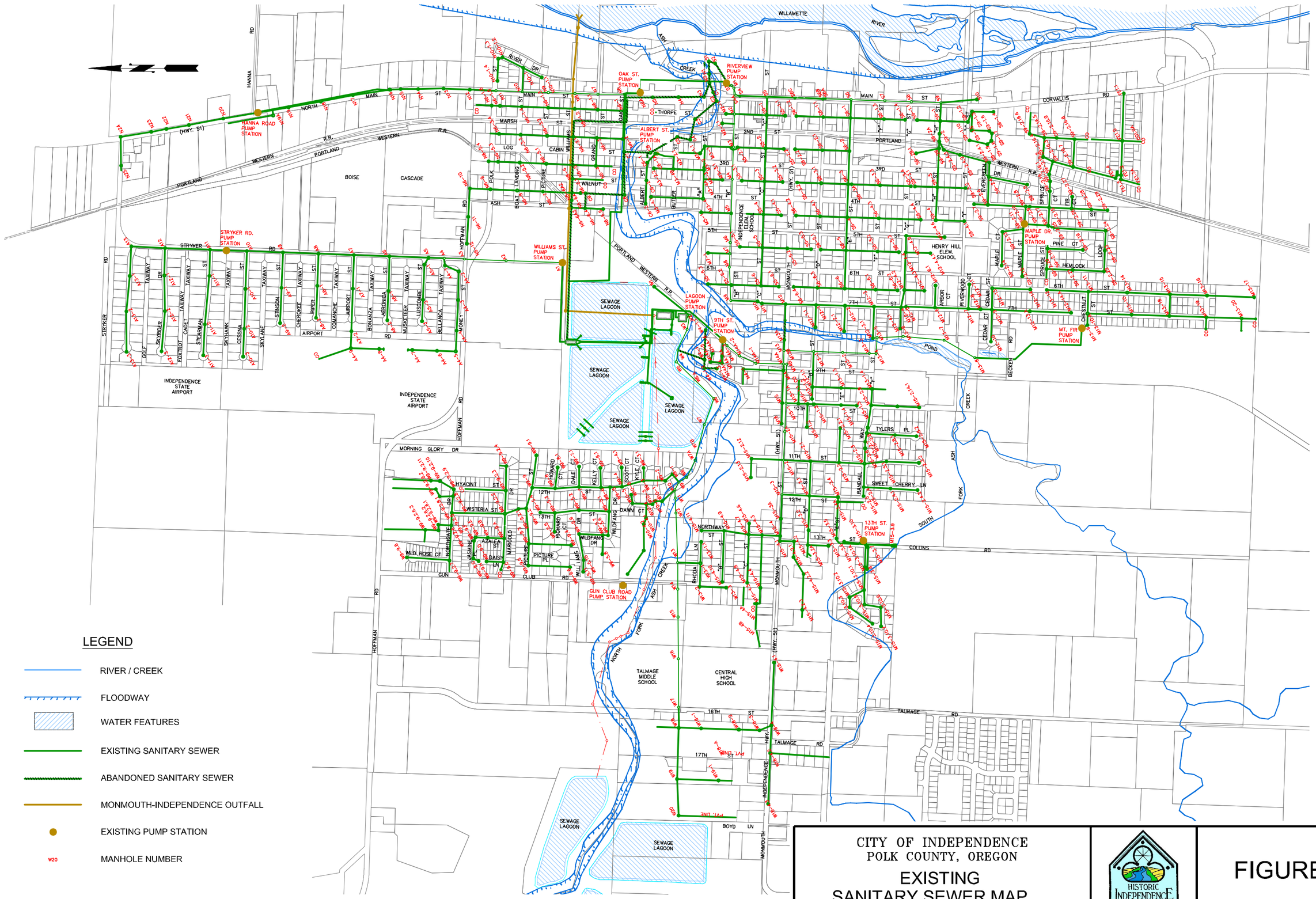
The collection system was originally configured to mimic natural drainage ways through town and flow by gravity toward the confluence of Ash Creek and the Willamette River. Although this approach is intended to be efficient in terms of energy use, it required that a

number of facilities be located adjacent to streams, within flood plains, and subject to high water and potential damage. The net result was that very large amounts of clean water were introduced to the waste treatment flow during flood events.

In the 1960's, the relocation of treatment facilities required that the cumulative flows delivered by gravity to the Riverview treatment facility, be redirected to a location approximately 4,700 feet west of this confluence of the influent sewers. As a result more energy was required to pump influent from the original primary treatment plant to the WWTF lagoons.

During the preparation of the *1996 Preliminary Design Report*, the entire approach to collection and treatment was reviewed and evaluated in the context of what was the most appropriate, efficient and environmentally responsible approach to managing wastewater. As a result, major reconfiguration of the collection system was completed which eliminated several old interceptors vulnerable to flooding, increased the hydraulic capacity and energy efficiency of key interceptors and pump stations, redirected some basins to follow a shorter path to the WWTF, and lowered the hydraulic grade line for key interceptors. Figure 3.1 provides a summary of the existing collection system configuration.

As part of the City's *1998 Sanitary Sewer System Improvement Project*, reconfigured collection system basins were established. Sewage is conveyed within the basin by gravity sewers and small lift stations to a basin influent pump station. Each influent basin pump station conveys the sewage through pressure pipes directly to a junction box and parshall flume at the wastewater treatment facility.



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

EXISTING
SANITARY SEWER MAP

HISTORIC INDEPENDENCE

FIGURE 3.1

3.1.2 Existing Sewer Basins

The existing collection system is subdivided into five basins, labeled alphabetically A through E. Each basin pump station then pumps the wastewater via pressure mains to the headworks of the WWTF. A detailed description for each basin is included in the following pages. Figure 3.2 is a map provided at the end of this section (3.1.2) that shows the location of each basin within the existing sewer system.

3.1.2.1 Basin 'A'

Basin A is located in the northeast corner of the City and includes residential, commercial and industrial land uses. Pipe materials are primarily concrete pipe with mortar joints, and vitrified clay. The south half of the basin is a portion of the City referred to as “Old Town” and is served by some of the oldest segments of the collection system. Much of the Old Town collection system is in poor condition and known to contribute large amounts of I/I.

The North Interceptor is the primary interceptor serving Basin A, and runs North to South in Hwy 51. The North Interceptor ranges in pipe diameter from 8 inches to 15 inches and discharges into the Oak Street Pump Station.

The upper end of the basin begins at the intersection of Stryker Road then flows south to the Oak Street Pump Station. There is one small pump station (Hanna Road Pump Station) located at the far north end at the intersection of Main Street and Hanna Road. The Hanna Road Pump Station was replaced in 2006 as part of the Hwy 51 improvement project.

During average storms this area experiences moderate to heavy I/I with the heaviest amounts contributed by the sub-basins in the vicinity of Riverview Drive on the east side of Main Street, and the “Old Town” area which is includes Walnut, Log Cabin, and Boat Landing streets. The segments furthest downstream in the basin and located within the 100-year flood plain, contained heavy root masses and were abandoned as part of the 1998 improvements.

Since 1998 city staff have completed several sewer rehabilitation projects primarily focused on the “Old Town” portion of basin.

3.1.2.2 Basin 'B'

Basin B consists of the southeast corner of the City including the downtown commercial area and numerous areas of residential development. Although this is an older portion of town, record drawings prepared by Westech Engineering indicate that a limited number of main lines serving the older parts of Basin B were replaced with plastic pipe in the early 1980’s as a pilot project. Unfortunately, the associated service laterals were not replaced at the same time as the mainline and field measurements indicate that the work had little measureable impact at reducing I/I from these areas. With the exception of some relatively

small pockets, this area is built-out with paved street improvements that include curbs, storm sewers and catch basins.

All of Basin B drains to the South Interceptor which runs along the east side of Main Street beginning at I Street and proceeding to the north before discharging to the Riverview Pump Station in the vicinity of Main and B Streets. The basins that are tributary to this interceptor have been the subject of a great deal of I/I investigative work. Smoke testing completed in August of 1996 identified numerous connected catch basins and roof drains that were subsequently disconnected as part of the 1998 improvement project. As part of that project, service laterals that were identified as faulty during the 1996 smoke testing were also replaced.

As part of the 1998 improvement project, a small submersible pump station was constructed at the north edge of the basin (Albert Street Pump Station). The Albert Street Pump Station allowed for the elimination of the original Creek Interceptor that was located adjacent to Ash Creek and known to contribute significant amounts of inflow, including direct inflows from Ash Creek. The Albert Street Pump Station serves approximately 8 homes with additional reserve capacity.

There are two additional, relatively small sewage lift stations, Maple Street and Briar Pump Stations. Maple Street Pump Station is located in a landscaped area adjacent to Maple Street and is a small package pump station which discharges to a 4-inch diameter force main conveying flows one block east to a manhole in the intersection of Maple and 4th Streets. Flow from this discharge manhole is conveyed by gravity to the South Interceptor.

The Briar Lift Station is small submersible station that serves approximately 50 homes that are part of the Freedom Estates development constructed in 2003. This station lifts sewage and conveys it approximately 50 feet to the Briar Road before discharging to the gravity sewer system.

In the *1996 Preliminary Design Report* the South Interceptor was identified as very close to capacity. However, key bottlenecks in the gravity line were replaced with larger diameter pipes as part of the 2002 Main Street Streetscape project and the upgraded interceptor now has increased hydraulic capacity to accommodate projected future flows. However, it should be noted that during extreme precipitation events the mainline is reported to surcharge. Several days of heavy rain in the winter of 2011 resulted in sewage flooding the basement of City Hall leading. As a result the City has installed backflow and isolation valves to prevent this from happening during future extreme events. No direct overflows or other basement flooding have been reported within this basin since the 1998 system wide improvements.

3.1.2.3 Basin 'C'

Basin C, in the City's southwest area, includes residential and commercial/retail land uses. Residential land within this basin is a mix of older neighborhoods and some of the newest developments in the City. Mt. Fir Subdivision, which is located at the southeast corner of the basin, is one of the most recent developments.

At full build-out the Mt. Fir property will extend far enough that an additional major pump station will be required. The original Mt. Fir Pump Station was constructed in 1999 and has provisions for expansion to add a third submersible pump if future growth or annexation requires. The station pumps flow generated in sub-basin C2, to a discharge manhole approximately 1,500 feet to the north. As part of the pump station improvements, the gravity sewer immediately downstream from the discharge manhole was replaced to increase hydraulic capacity.

The southwest corner of Basin C is served by the 13th Street Pump Station, which is a small submersible pump station. The 13th Street Pump Station is sufficiently sized to accommodate current flows from this relatively small sub-basin but has limited residual capacity and no provisions for expansion. All flows from Basin C are directed to the 9th Street Pump Station, which pumps directly north to the WWTF via a 10-inch diameter forcemain.

3.1.2.4 Basin 'D'

Basin D, in the City's West Side, incorporates a wide range of land uses including residential, commercial retail, general commercial, and school/institutional. Most of the basin is served by gravity sewers including the West Interceptor which follows an alignment that is roughly in the geographic middle of the basin. This interceptor flows from west to east in an alignment approximately parallel to Ash Creek and crossing under the creek from the south side to the north side approximately ¼ mile west of the WWTF. The West Interceptor discharges into the Lagoon Pump Station located at the southeast corner of the WWTF from which flows are pumped via a 10-inch force main to the influent vault and parshall flume at the north end of the WWTF.

In 2009 a new pump station (Gun Club Road Pump Station) was constructed and sized to serve the northwest corner of Basin D including land recently annexed into the city limits and an expansion of the urban growth boundary (UGB).

Much of the basin is concrete pipe with a mix of mortar and rubber gasket joints. The Northgate residential subdivision at the northeast corner of Basin D is relatively new construction that included PVC pipe and rubber gasket joints, and pre-cast manholes. This development is believed to be relatively "watertight". The older portions of the collection system, including the Wildfang Subdivision immediately south of Northgate, include construction and materials reported to be leaking and contributing significant quantities of

I/I. While completing field investigations in preparation for the 1998 Sewer System Improvements, relatively clear flows were noted in the far west segments of the West Interceptor during heavy rainfall events. However, subsequent smoke testing did not identify specific faults in service laterals, storm water or roof drain connections. The conclusion is that the contribution is primarily systemic infiltration resulting from cumulative small flows associated with numerous joints and small faults in the collection system that will be very expensive to correct.

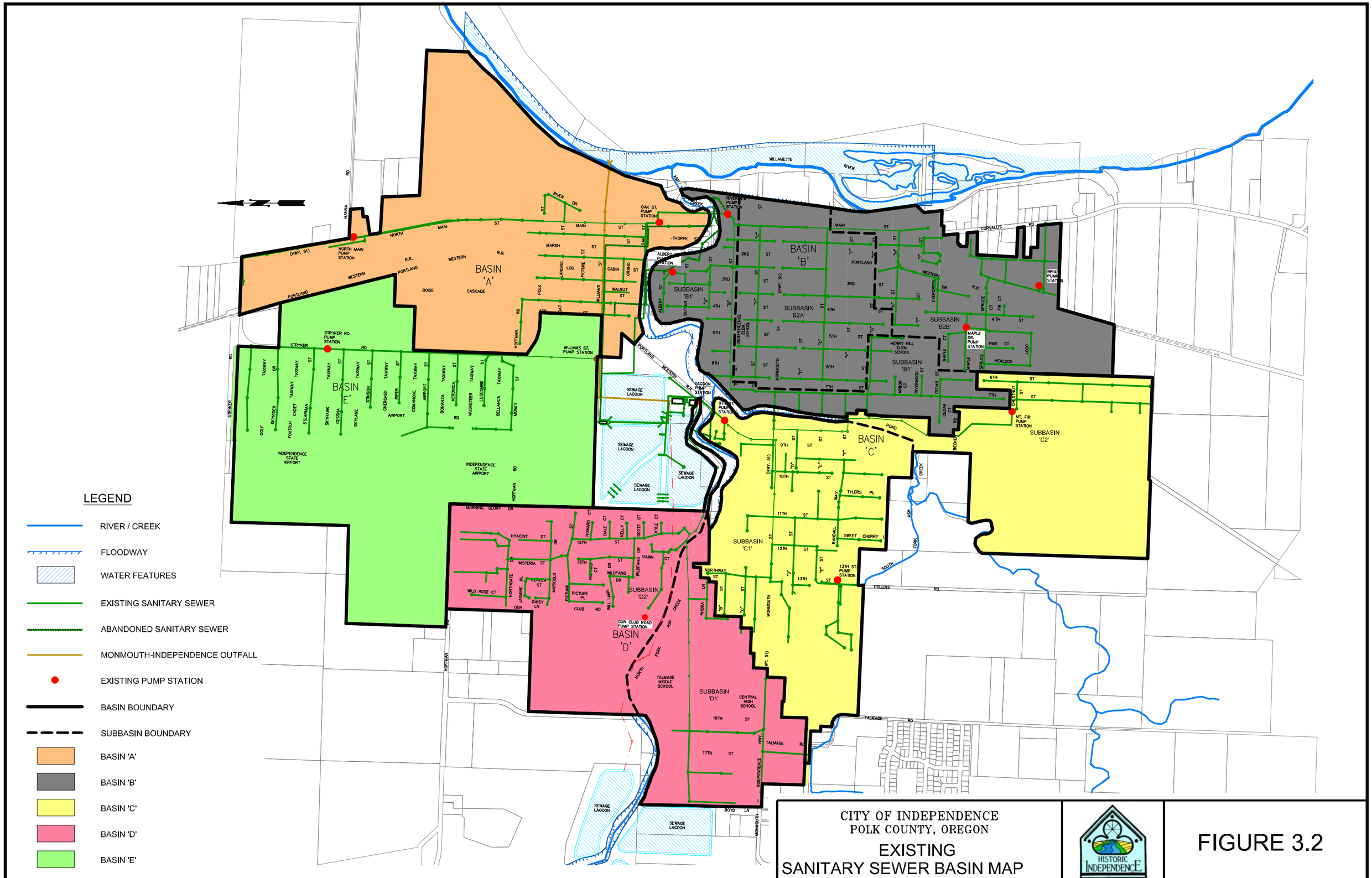
The current urban growth boundary includes approximately 106 acres of developable land that was recently added at the northwest corner of the current basin limits. In response to pressure by private interests to develop residential properties west of Gun Club Road, a new pump station was constructed in 2009. Although gravity sewer has not been extended to the boundaries of this basin, the new pump station has been constructed at a depth and with a configuration that is anticipated to allow for capacity to serve ultimate development in the basin. Depending on the nature of development and utilities, an additional pump station may be required to serve all of the properties north of Hoffman Road which are currently included in Basin E.

3.1.2.5 Basin 'E'

Basin E is north of the City's sewage lagoons and includes the airport and airpark residential area. Land uses include residential, commercial, and industrial properties. Most of the residential development in the basin is served by PVC pipe with rubber gasket joints. The industrial properties could be redeveloped and characteristically have the potential for development with significantly higher water use and subsequent wastewater generation.

The north end of the Airpark residential area is served by a small package (wetpit/drypit) pump station (Stryker Road Pump Station). This station serves an estimated 78 residential lots and potentially 22.6 additional acres of MX land at the north end of Stryker Road. The cumulative flows in the basin discharge to the Williams Street Pump Station, which is, located immediately northeast of the WWTF. The Williams Street Pump Station is a submersible configuration which pumps to the influent equalization vault at the north end of the WWTF via a 6-inch forcemain. In the fall of 2013 one of the pumps failed and the Williams Street Pump Station was upgraded with new Flygt submersible pumps and new electrical equipment. The original metal structure remains in use. The Williams Street Interceptor and pump station will require replacement with larger facilities to accommodate projected future flows. As noted above, the development of industrial properties in this basin could dramatically impact the peak wastewater flows. For this reason planning and design of new or replacement sewerage facilities should include consideration for expansion and phasing.

The location and configuration of the Existing Sewer Basins are shown in Figure 3.2.



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

EXISTING
SANITARY SEWER BASIN MAP

FIGURE 3.2

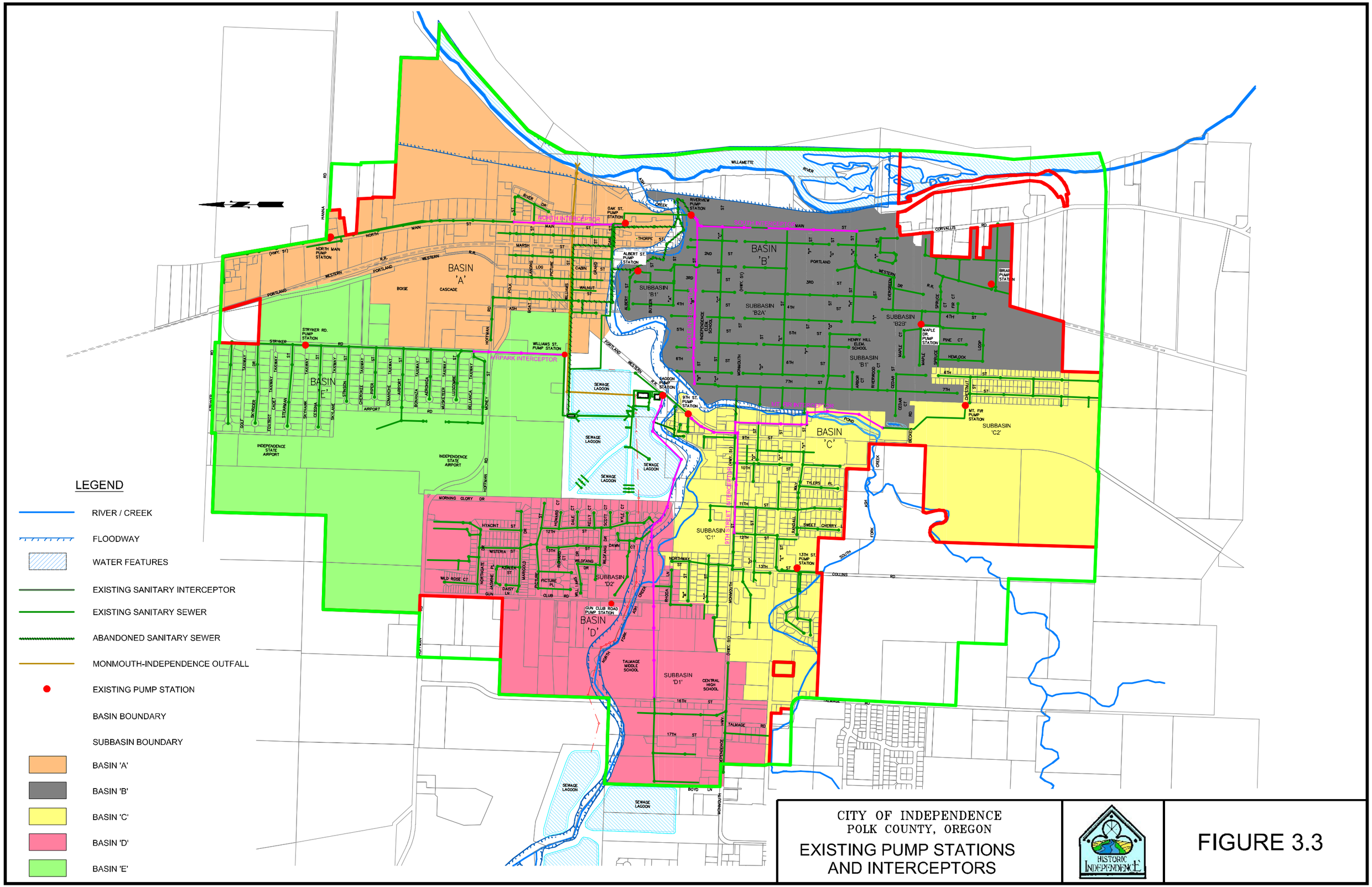
3.1.3 Existing Pump Stations and Interceptors

Background information including capacities, configurations, size, depth and slopes of many of the existing interceptors, pump stations and force mains was obtained from a variety of records made available by City staff. While this information is believed to be representative of the current configuration, all key components should be confirmed as part of the preliminary design effort in support of any future improvements.

Major interceptors within the service area consist of pipe sizes ranging from 10 to 21 inches in size. The pump and lift stations range in size from 147 gallons per minute (gpm) to 2,250 gpm, with force main diameters ranging from 4-inches to 16-inches. All pump stations have either dedicated stand-by auxiliary power generators, or a connection for a portable generator. Table 3.1 summarizes the pump station and force main information. The location of existing pump stations and interceptors are shown in Figure 3.3.

Table 3.1 Pump Station Inventory

Pump Station Name/Location/.Date	No. of Pumps	Horse Power Voltage, RPM, Aux. Power	Rated Capacity and Classification	Type of Pumps	Level Controls	Comm Type	Force Main
Riverview Pump Station Main St. and B St. Constructed 1998	2	60 Hp, 460V, 3Ø 1770 rpm Aux. Power = 125 kW	2,250 gpm @ 65' TDH (Riverview Only), EPA Class I 2,100 gpm @ 69' TDH (with Oak St.)	Flygt Model CP3300.090-644 HT	Flygt Multitrode	Auto Dialer	1,162 lf 14" C905 3,611 lf 16" C905 to WWTF Flume
Oak Street Pump Station Oak St. and Main St. Constructed 1998	2	20 Hp, 460V, 3Ø 1750 rpm Aux. Power = 60 kW	1,070 gpm @ 45' TDH (Oak St. Only), EPA Class I 740 gpm @ 59' TDH (w/ Riverview)	Flygt Model CP3152.091-432 MT	Flygt Multitrode	Auto Dialer	50 lf of 8" C900 3,611 lf of 16" C905 to WWTF Flume
9th St. Pump Station Constructed 1998	2	20 Hp, 460V, 3Ø 1750 rpm Aux. Power = 60 kW	900 gpm @ 65' TDH, EPA Class I	Flygt Model CP3152.091.432 MT	Flygt Multitrode	Auto Dialer	2,160 lf of 10" C900 and DIP to WWTF Flume
Lagoon Pump Station Remodeled 1999	2 w/ capacity for 3	40 Hp, 460V, 3Ø 1185 rpm Aux. Power = 125 kW	1509 gpm @ 65' TDH, EPA Class I	Flygt Model CP3201.090-639MT	Floats	Auto Dialer	1,493 LF of 10-inch C-900 PVC to WWTF Flume
Williams St. Pump Station Constructed 1979 At Lagoon Entrance	2	10 Hp, 230V, 3Ø	425 gpm @ 56' TDH, EPA Class II	Flygt Model NS 3127 HT3	Floats	AutoDialer	680 Lf of 6"to WWTF Flume
Albert St. Pump Station Albert St. and Log Cabin Constructed 1998	2	5 Hp, 460V, 3Ø 1150 rpm	147 gpm @ 22.3' TDH, EPA Class II	Hydronix S4MVX, 8.5" Dia. Impeller	Floats	Auto Dialer	342 lf of 4" C900
Hanna Road Pump Station OR 51 and Hanna Road	2	5 Hp, 1755 rpm	250 gpm @ 29' TDH, EPA Class II	Flygt NP-3102-464	Flygt Multitrode	Auto Dialer	1,678 lf of 6", C-900 PVC
13th Street Pump Station 13th and E St. Modified 1998	2	3 Hp, 230V, 3Ø	192 gpm @ 26.4' TDH, EPA Class II	Flygt NP 3085,P92, 410 impeller	Ultrasonic	Auto Dialer	320 lf of 4" Cast Iron
Maple St. Pump Station Maple Dr. and Maple Ct. Modified 1998	2	2 Hp, 230V, 1150 rpm	250 gpm @ 15' TDH, EPA Class II	Hydromatic Submersible 7.54" Impeller	Floats	Auto Dialer	170 lf of 4"
Mt. Fir Pump Station 7th and Chestnut St. Constructed 1999	2 w/ Capacity for 3	7.5 Hp, 230V, 3Ø 1750 rpm	568 gpm @ 20' TDH (1 Pump) 700 gpm @ 70' TDH (2 pumps), EPA Class I	Flygt MP-3127	Ultrasonic	Auto Dialer	1,450 lf of 8" C900
Stryker Rd. Pump Station Stryker Rd and Cessna Street	2	3 Hp, 230V, 1750 rpm	155 gpm @ 20' TDH, EPA Class II	Hydromatic Model 40 MPV	Floats	Auto Dialer	312 lf of 4-inch C900 PVC
Briar Rd Pump Station Briar and Alder St. Constructed 2003	2	3 Hp, 230V, 1Ø 1750 rpm	200 gpm @ 22' TDH, EPA Class II	Flygt NP3085.092-462	Ultrasonic	Auto Dialer	50 lf of 4" Ductile Iron
Gun Club Road Pump Station Gun Club Road Constructed in 2009	2	20 Hp, 460V, 3Ø, 1750 rpm Aux. Power = 60 kW	525 gpm@74' TDH, EPA Class I	Flygt NP3153.091 HT	Flygt Multitrode	Auto Dialer	1,947 lf or 6" C900 PVC



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

EXISTING PUMP STATIONS AND INTERCEPTORS

FIGURE 3.3

3.2 Wastewater Treatment Facility

In the early 1950's, treatment of the City's sanitary sewage was provided by a primary clarifier constructed within the 100-year flood plain at the confluence of Ash Creek and the Willamette River. The original plant provided primary wastewater treatment for both Monmouth and Independence.

In the 1960's, in response to increasing population, periodic overflows, more stringent treatment standards, and the plant's location in the flood plain, the cities elected to separate their wastewater collection and treatment systems, and constructed separate lagoon treatment facilities. Independence constructed what are now Cell Nos. 2 and 3 in the northwest corner of the current WWTF.

In 1978, to provide additional lagoon volume to accommodate a higher population, the City lagoons were expanded by adding two more cells. The cells constructed in 1978 are currently labeled Nos. 1 and 4. Chlorine disinfection was improved with construction of a 1-ton cylinder chlorine storage room, chlorinators, injectors, and an 82,700 gallon chlorine contact basin. The City's treated effluent then discharged to the 36-inch Independence-Monmouth effluent outfall that was constructed at the same time as the lagoon upgrade.

In 1998, the WWTF was upgraded to provide a centralized point of collection, influent flow measurement, and improve flow control to lagoons. Influent flow measurement allowed the plant to better manage peak flows associated with I/I. The flow control improvements gave operators more flexibility in operation, and eliminated problem areas.

Today, the WWTF is virtually unchanged from 1998. Treated effluent from the two cities continues to be discharged to the Willamette River during permitted wet weather months, via the common 36-inch-diameter river outfall built during 1978.

3.2.1 Description

The Independence WWTF is a 4-cell, controlled discharge, facultative lagoon system with chlorine disinfection and discharge to the main stem of the Willamette River. The lagoons can be operated in series or with cell Nos. 1 and 2 functioning in parallel. Detention time is maximized when operating in series (1 through 4).

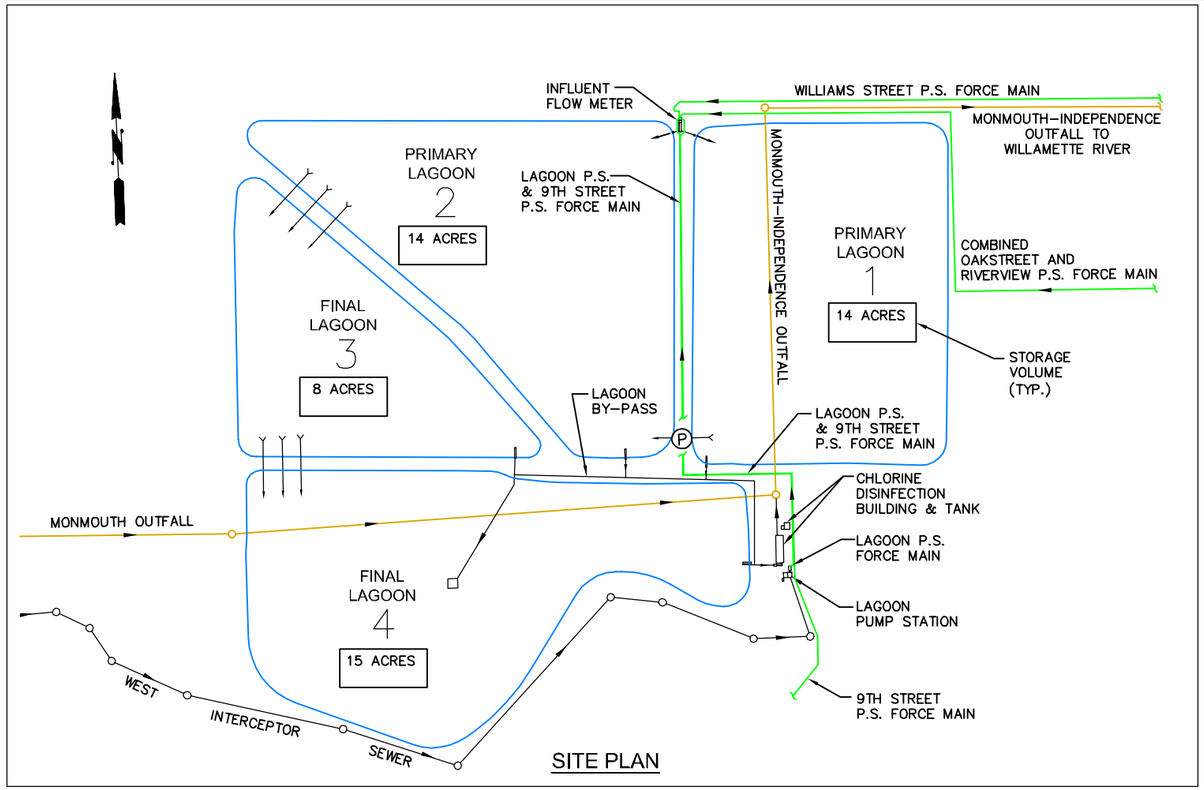
Figure *3.4 Existing Wastewater Treatment Facility* is a diagrammatic representation of the facility showing both schematic and plan views. Table 3.2 provides a summary of current design parameters.

Table 3.2 Existing Treatment Facility Design Data (As Designed)

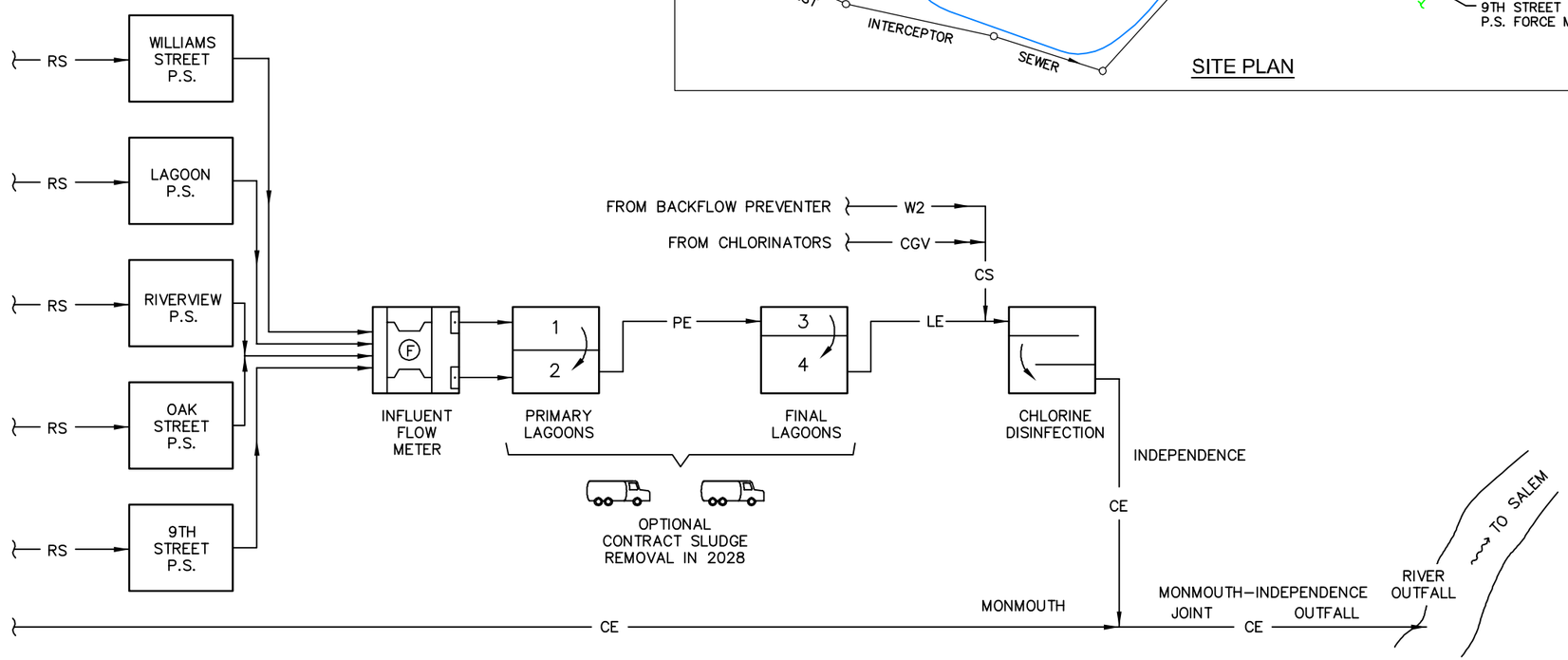
Headworks				
Flow Stabilization Vault	11.0 mgd			
Parshall Flume Size	12 inches			
Parshall Flume Range	0.23 – 11.0 mgd			
Flow Splitter Structure	11.0 mgd			
Facultative Lagoons				
	Lagoon No. 1	Lagoon No. 2	Lagoon No. 3	Lagoon No. 4
Minimum Water Elevation	170’	175.7’	170’	167’
Maximum Water Elevation	175’	180.3’	174’	171.6’
Usable Storage Volume (acre - feet)	70	64	32	69
Surface Area at Average Depth (acre)	14	14	8	15
Discharge Period				
Summer Holding Period:	June 1 through October 31			
Winter Discharge Period:	November 1 through May 31			
Disinfection				
Type:	Chlorine Gas Injection			
Capacity:	210 feet long, 3.0 mgd			
Detention Time:	1 hour including outfall			
Parshall Flume:	9 inch, 3.0 mgd			
Outfall Discharge				
36-inch Shared Outfall with the City of Monmouth to the Willamette River				
Material:	Concrete Pipe			

LEGEND

CE	CHLORINATED EFLUENT
CGV	CHLORINE GAS VACUUM
CS	CHLORINE SOLUTION
DS	DIGESTED SLUDGE
F	FLOW METER
FM	FORCE MAIN
LE	LAGOON EFLUENT
(P)	PUMP
PE	PLANT EFLUENT
RAS	RETURN SLUDGE
RS	RAW SEWAGE
SDS	SULFUR DIOXIDE SOLUTION
WAS	WASTE SLUDGE
W1	WATER – POTABLE
W2	WATER – PROCESS
W3	WATER – IRRIGATION
3/3	BOD AND SOLIDS IN mg/L



SITE PLAN



SCHEMATIC

CITY OF INDEPENDENCE
POLK COUNTY, OREGON

EXIST. LAGOON TREATMENT SITE PLAN & SCHEMATIC

FIGURE 3.4

3.2.2 Current NPDES Permit Requirements

Beginning with the preparation of the 2005 Wastewater Master Plan members of the consultant team met with representatives of the Oregon Department of Environmental Quality (DEQ). During the preparation of this Facilities Plan DEQ staff confirmed that the City's NPDES permit remains unchanged since the 2004 renewal. However, DEQ staff noted that the anticipated changes that will be part of the next permit review and renewal cycle will likely include additional monitoring requirements and potentially more stringent pollutant limitations.

A copy of the current NPDES permit is included as Appendix A of this report. The permit allows discharge from an outfall to the Willamette River located at R.M. 95.5. Table 3.3 NPDES Permit Requirements – Treatment Standards summarizes the Biological Oxygen Demand ($BOD_{5)}$, Total Suspended Solids (TSS) and bacterial limits set by the City's permit. Table 3.4 NPDES Permit Requirements – Monitoring Requirements summarizes the parameter, frequency, and method of monitoring set by the City's permit.

Table 3.3 NPDES Permit Requirements - Treatment Standards

	Avg. Effluent Concentration		Mass Load Limits		
Parameter	Monthly	Weekly	Month Avg.	Weekly Avg.	Daily Max.
BOD	30 mg/l	45 mg/l	500 lb/day	750 lb/day	1,000 lb/day
TSS	50 mg/l	80 mg/l	830 lb/day	1,200 lb day	1,700 lb/day

E. Coli Limits

E. Coli Bacteria shall not exceed 126 organisms per 100-ml monthly geometric mean.
No single sample shall exceed 406 organisms per 100mL.
pH shall be in the range of 6.0 – 9.0.
BOD_5 and TSS Removal efficiency shall not be less than 85% monthly average for BOD_5 and 65% monthly for TSS.
Total Residual Chlorine shall not exceed 1.0 mg/l daily maximum.

Outfall Discharge Requirements

Discharge from the outfall is permitted to the Willamette River during the time period from November 1 through May 31. The allowable mixing zone for the outfall extends fifty (50) feet from the west bank of the river and extends from a point fifty (50) feet upstream from the outfall, to a point three hundred (300) feet downstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within thirty (30) feet of the point of discharge.

Monitoring Requirements

Under Schedule B of the permit the City must monitor and report influent flows and loads:

Table 3.4 NPDES Permit - Monitoring Requirements

Parameter	Minimum Frequency	Type Of Sample
Total Daily Flow (mgd)	Daily	Measurement
Flow Meter Calibration	Annually	Verification
BOD_5	Weekly	Composite
TSS	Weekly	Composite
pH	3/Week	Grab
Treated effluent must be monitored and reported for the following:		
Total Daily Flow (mgd)	Daily	Measurement
Flow Meter Calibration	Annually	Verification
BOD_5	Weekly	Composite
TSS	Weekly	Composite
pH	3/Week	Grab
E. Coli	Weekly	Grab
Quantity of Chlorine Used	Daily	Measurement
Chlorine Residual	Daily	Grab
BOD_5 & TSS Pounds Discharged	Weekly	Calculation
BOD_5 & TSS Average % Removed	Monthly	Calculation
Iron	Annually	Grab
Mercury	Annually	Grab
Dissolved Oxygen	Monthly	Grab
Ammonia	Monthly	Grab
Effluent Temperature Daily Max.	2/Week	Grab
Excess Thermal Load	Weekly	Calculation

Iron and mercury monitoring shall occur annually in January. Dissolved oxygen, ammonia and effluent temperature shall be monitored only when discharging. Excess thermal loading shall be monitored from October 15 through May 15.

3.2.3 Headworks

All flow delivered to the WWTF is pumped through one of four force mains. All solids conveyed to the plant pass through a pump station wet wells and pump impellers that physically prevent solids larger than 3 inches in diameter from entering the influent wastestream. For this reason the current treatment facility has minimal influent management facilities consisting of measurement, sampling and flow control.

The major components of the headworks consist of a concrete flow stabilization vault, a 12-inch Parshall flume, and ultrasonic flow measurement device. Instantaneous flow measurement is recorded on a “circle” chart recorder and total accumulated flow is recorded by a flow totalizer then logged daily by the operator. The Parshall flume has a staff gage for manual measurement and calibration of the flow meter. An influent composite sampler is adjacent to the Parshall flume in a vandal proof pre-fabricated fiberglass structure. The sampler automatically collects wastewater that is sent to a certified lab for testing of required strength and wastewater characteristic parameters.

All of the existing force mains discharge to the WWTF at the concrete influent stabilization structure upstream of the Parshall flume. Flows pass from the stabilization structure via an 18-inch gravity pipe flowing into the Parshall flume. After passing through the Parshall flume, the flow stream enters a flow splitter configured with sluice gate valves to direct influent to either Lagoon No. 1 or 2.

3.2.4 Sewage Treatment Lagoons

The City lagoon treatment system is a “facultative” lagoon process, involving different microorganisms in the upper, middle and bottom layers. The City ponds are considered “facultative” because they are relatively shallow and have an anaerobic (oxygen-deprived) zone at the bottom, an aerobic zone at the top, and an intermediate aerobic/anaerobic zone in between. Each zone supports different types of microorganisms that convert the waste to cell matter and biochemical products.

Although the organisms can only survive in their zone of origin, the three zones are symbiotic – work together – in removing the soluble BOD from the upper zones and, on the bottom, digesting the organic part of the sludge.

Much of a lagoon’s oxygen budget is from photosynthesis of algae during the daylight hours. At night, algae cells are consumed by bacteria, which in turn produce carbon dioxide. The carbon dioxide is used for the following day’s production of new algae cells. Although algae is important to a lagoon’s operation, at the effluent end, the small 1-8 micron cells are very difficult to remove and can cause elevated levels of effluent solids and BOD.

Lagoons have some advantages over “mechanical” treatment facilities. Properly sized and configured lagoons provide BOD removal at rates of 10 to 50 lbs/day per acre with an overall removal efficiency between 65 and 90 percent. They also provide excellent settling of solids during all flow conditions. Lagoons, unlike mechanical plants, are resistant to upset from sudden inflow. The lagoon’s capacity to store wastewater during the low-flow or “no-flow” season helps operators stay within the permitted loading limits by reducing the effluent flow as required. In areas where land is flat and inexpensive, lagoons are often the least costly means of wastewater treatment for small cities. They are very easy to operate, and generally require only a Level I operator certification.

On the other hand, lagoons have some disadvantages. They can reliably achieve secondary standards of 30 mg/l effluent BOD or 85 percent BOD removal but this is not as efficient as activate sludge or other “mechanical” treatment plants. Algae is a principal problem, sometimes making it difficult to achieve effluent solids below 30 mg/l during seasonal algae blooms. Since more lagoon area is needed to increase storage and treatment capacity, space, competing uses, and land cost become issues as a community grows.

Lagoon Operation – Wastewater typically flows sequentially through the lagoon cells, although the transfer pump and overflow pipes allow for different flow patterns if conditions dictate. To operate the facility, the operator calculates influent and effluent flows, pollutant loads, and the chlorine feed rate. Influent flows are measured at the headworks. The magnitude of effluent flow is determined on the basis of BOD and solids removal requirements, and the need to store or discharge effluent. The chlorine feed is a function of the effluent flow and quality, and bacteria sampling data.

If a lagoon is too shallow, the light and warmth in the water column can increase algae blooms. The Transfer Pump and outlet structures are used to maintain sufficient depths in the lagoons to optimize effluent quality.

Lagoon cell Nos. 1 and 2 typically serve as the primary cells with cell Nos. 3 and 4 providing secondary treatment. As a result of the phased approach to constructing the lagoons, the bottom and the operational minimum water elevation of cell No. 1 is approximately 6-feet below that of cell No. 2. Although it appears to be possible to transfer partially treated wastewater directly from cell No. 1 to the final cell No. 4, historically, public works staff transferred much of the partially treated wastewater from lagoon No. 1 to No. 2 via a transfer pump that is located at the southwest corner of cell No. 1. The transfer pump is a single pump and is not capable of transferring large volumes of waste in a short period of time. Although staff must monitor and balance the levels of the two primary cells, the operational flexibility of the larger system is sufficient that the capacity of the transfer pump has not been a major concern or bottleneck in the past.

There are a number of gravity type transfer and overflow pipes connecting the cells, which allows for a wide range of operational configurations. With the exception of the pipe connecting cell Nos. 3 and 4, all transfer pipes connecting lagoon cells are configured with valves to allow for variable flow transfers and/or isolation. As part of the 1998 improvement project a new transfer and flow control structure was built between cell Nos. 2 and 3. The transfer structure includes an inlet configuration at cell No. 2 that utilizes flexible 18-inch diameter reinforced rubber flex hose that can be adjusted in elevation to control the flow rate and decanting depth. In addition to this transfer structure, staff gages were installed at all four lagoon cells to allow for the measurement and monitoring of lagoon depths.

Wind causes wave action within the lagoons and requires riprap to be placed at the edges to reduce the potential for erosion. Periodically additional riprap needs to be placed replacing eroded material. During the summer of 2004 the City Public Works crews installed approximately 2,300 tons of riprap around the perimeter of all four lagoons. Greater quantities were placed at the southern edge of each cell to reduce the erosion potential caused by prevailing wind.

3.2.5 Chlorine Disinfection System

The WWTF's chlorine system relies on one-ton chlorine cylinders, which are loaded and unloaded with the monorail and hoist in the Chlorine Bay. For safety, chlorine is "pulled" from the cylinders by a vacuum created by the "chlorine injectors," causing flow to stop in the event of a pipe failure. The injectors mix water with the chlorine to produce a strong chlorine solution that is mixed into the effluent flow through a perforated-pipe chlorine "diffuser" located in the flowstream. An effluent valve downstream of the diffuser was the original chlorine-mixing device, although this valve is unused at this time.

After the effluent is mixed with chlorine, it discharges to the chlorine contact basin, where sufficient time is permitted for the chlorine to kill the bacteria prior to discharge and dilution in the effluent outfall. A chlorine contact time of at least 60 minutes is required, but the actual time is considerably greater for most flow conditions.

The amount of chlorine needed for an acceptable kill is affected by algae solids and BOD_5, and by the state of the nitrogen in the discharge of Lagoon 4.

The equipment in the chlorine injection system is nearing the end of its operational life and replacement of key components is assumed. In the coming years it is likely that the issue of chlorine toxicity at the discharge to the Willamette River will become a focus of permit discussions. An updated Mixing Zone Study and model for the outfall shared by the Cities of Independence and Monmouth is currently under review and will help to determine if additional measures are required. With these factors in mind it has been assumed that disinfection using new chlorination/dechlorination facilities will be required under all treatment alternatives considered. If the Mixing Zone Study determines otherwise, dechlorination facilities can accordingly be removed from the recommended capital improvements.

3.2.6 Irrigation Set

To eliminate some of the stored water during the dry weather storage season, plant operators deploy a reel-type spray irrigation set that sprays treated and disinfected effluent over the surface of Lagoon Cell No. 1 and land surfaces immediately adjacent to the cells. Large setbacks from residential property lines are required for this type of spray irrigation, and operators have placed it along the lagoon margins. Operators report that evaporation of the spray helps to reduce the stored volume. This type of irrigation is likely exempt as set forth in Division 55 if it is limited to the treatment plant property.

3.2.7 Willamette River Outfall

The City's disinfected effluent is discharged to the joint effluent outfall at the manhole just north of the chlorine contact basin. Flow enters the manhole from the City's 15-inch discharge pipe and Monmouth's 24-inch, 10,750-foot-long RCP outfall pipe. Exiting the manhole is the 36-inch RCP Independence-Monmouth effluent outfall. The 4,500-foot outfall proceeds north under Lagoon 1 then east along Williams Street to the bank of the Willamette River. From the discharge manhole on the bank, a sloped, 24-inch, 50-foot-long outfall pipe extends to a spill pad at the bottom of the river.

In 1995 the Cities of Independence and Monmouth completed at Mixing Zone Study for the Wastewater Treatment Facility Outfall. The study was performed to determine if the residual chlorine discharges meet water quality requirements in the Willamette River. The results of the study indicated that the chlorine residual was negligible at the outfall due to the transport time of the effluent between the WWTF and the Willamette River.

At the time this Facilities Plan was submitted a draft updated mixing zone study has been prepared by Kennedy Jenks and submitted to DEQ.

It should be noted that the outfall location underneath Lagoon Nos. 1 and 4 limits construction options should the outfall require repair or replacement. There is no indication that the outfall pipe underneath either lagoon was not soundly constructed. However, the pipe is currently 37 years old and will eventually require maintenance or repair. City Public Works staff reported that within the last eight years the pipeline has been inspected using internal TV equipment with no structural problems observed.

3.2.8 Treatment Performance

The performance of the existing WWTF was evaluated by reviewing plant flow and sampling data for the time period between January 2007 and December 2014. During the review of historic flows, loads and trends, it became apparent that the measurements of flows reported by the new influent flow meter beginning in January 2011 appeared to be significantly higher than in previous years. This general trend was constant through all seasons from January 2011 through December 2014.

The impact to the analysis and subsequent recommendations for future improvements is significant and the consulting team completed a focused attempt to clarify whether the difference in readings is due to problems with the new equipment, or if influent measurements before January 2011 were perhaps reported too low. Because the dramatic change in recorded flows occurred immediately after replacing the flow meter, it is suspected that meter calibration was the source of the discrepancy and not a change in population or other sources of wastewater.

Three approaches were applied in an attempt to identify the cause for the measurement discrepancies. These included: *Comparison of Total Annual Measured Influent Volume*

and Measured Effluent Volume, *Comparison of Total Measured Influent Volume with Total Estimated Pumped Volume*, and *Dry Weather Water Balance*. Of the three approaches the dry weather water balance of the lagoons produced the results with the highest degree of confidence. Based on this analysis it appears that the influent meter readings after January 2011 are a more accurate reflection of influent discharged to the lagoons. Since the installation of the new meter, City Public Works staff have calibrated the meter on an annual basis through the services of an outside contractor.

Lagoon Water Balance

The following paragraphs describe the dry weather water balance completed. There are multiple variables involved in a traditional water balance with some degree of variance inherent in the precision of the measurements and equipment. Although it is impossible to eliminate these variances and potential errors, using data collected over multiple years helps to reduce the impact on results, and provides a check and balance regarding the order of magnitude of existing storage requirements.

Volume Change in Lagoon = (Volume Input) – (Volume Output)

$$V_{Change} = (V_{Influent} + V_{Precipitation}) - (V_{Effluent} + V_{Seepage} + V_{Evaporation})$$

Precipitation, evaporation, influent, effluent, and lagoon volume changes can be measured directly. However, the amount of seepage from the lagoons cannot easily be measured and must be calculated. Through a trial and error process it was determined that a seepage rates ranging between 0.20 – 0.29 inches per day produced volume changes consistent with measured volumetric totals for influent, precipitation, evaporation and effluent during years of early release. Typical infiltration values for lagoons constructed without plastic liners are expected to be in the order of 0.25 inch per day. When estimating the required additional volume for storage or recycled use, the lower the infiltration rate used in the water balance, the more volume that will remain in the lagoons and require storage or spray irrigation. A traditional 0.25 inch per day rate would fall within the range of data recorded during the observation period and provides a somewhat conservative assumption for estimating future management of treated effluent.

It has also been established through topographic surveys, review of original design drawings and two recent sludge surveys that the lagoons have a residual storage volume of 52 million gallons (MG).

In recent years including during the study period of 2011 and 2014, the City has requested and been allowed to discharge effluent early to the Willamette River to prevent overtopping of the lagoons. This trend and the water balance performed as part of this report would indicate that the existing lagoon storage volume of 52 MG is not sufficient to accommodate the current flows. Additional lagoon storage or an alternate method for managing summer flows must be developed.

Table 3.5 provides a summary of the volume that currently exceeds available lagoon storage. This number represents the calculated change in volume from the water balance, and the total effluent that was recorded to have been released prior to the permitted release date of October 31st. This amount is sensitive to precipitation totals during dry weather. During the observation period total precipitate varied between 4.44 and 8.87 inches. Average precipitation totals for June – October measured at the Corvallis, Oregon weather station total 7.7 inches/year. Interpolating between the minimum and maximum volume values of this range and applying the average precipitation predicts an annual average of 21 MG in excess of current lagoon storage. To capture unusually wet years an additional volume of 25 MG is recommended to serve the current (2015) population.

Table 3.5 Lagoon Water Balance – June 1 through October 31

Year	Total Influent(1) (MG)	Total Effluent(2) (MG)	Prec.(5) (MG)	Infiltr. (MG)	Evap.(6) (MG)	Estim. Change Volume (MG)	Lagoon Storage Capacity(4) (MG)	Required Additional Volume(3) (MG)
2011	147.0	-48.6	6.1	-53	-34.3	17.3	52	13.9
2012	153.8	-13.5	12.3	-53	-37.2	62.4	52	23.9
2013	143.5	-53.4	12.1	-53	-31.7	17.5	52	18.9
2014	141.5	-33.7	6.9	-53	-35.5	26.3	52	8.0

Assumed rate of Infiltration = 0.25 inches/day = 0.346 mg/day

Evaporation Pan Coefficient = 0.745

(1) Total Influent recorded and reported on DMR (June 1 – October 31)

(2) Total Effluent recorded and reported on DMR (Early discharge prior to October 31)

(3) Total calculated Water Balance based on measured influent, effluent, precipitation, evaporation and estimated seepage (June 1 – October 31)

(4) Total residual storage capacity of lagoons accounting for measured sludge accumulation.

(5) Total Precipitation recorded between June 1 – October 31 at lagoons.

(6) Total Evaporation recorded June 1 – October 31 at Corvallis Weather Station.

Based on the analysis and comparison described in the previous sections there is higher level of confidence in the influent flows measured after January 2011. Some design parameters, such as BOD_5 and TSS concentrations, are not believed to have been impacted. However, the BOD_5 and TSS influent mass loads are a direct calculation based on both concentration and influent flow and as such only those after January 2011 were used in estimating future loads.

3.2.8.1 Biochemical Oxygen Demand (BOD) Removal

Table 3.5 *BOD and Solids Removal, 2011 - 2014* summarizes the BOD and solids data and removal efficiencies for that period. The efficiencies, computed from the incoming

loads and the outgoing, "months-old" effluent, may be seen to be less than that required by the plant's DEQ permit. However, DEQ realizes that low percent removal efficiencies may sometimes be the result of dilute and abnormally low incoming loads at the time discharge is occurring, and that the presence of algae can mask the treatment performance.

Because of the algae's active photosynthesis and respiration, BOD is not a good indicator of plant performance. In other words, if the difficult task of removing algae from the effluent at the outlet of the lagoons could be accomplished the plant's BOD and solids removal performance could be substantially greater. Regardless, the DEQ permit requirements still apply.

Biochemical Oxygen Demand (BOD) - This parameter combines the oxygen need of all oxygen-using pollutants into a single parameter. The data indicates that between 2011 and 2014 the lagoons removed an average of 86.4 percent of the influent BOD. A monthly average 85-percent minimum BOD removal is specified in the DEQ permit. A subsequent review of the plant records indicates that on a month to month basis during the 2011-2014 timeframe there were several periods when the percent removal was below that required under the NPDES permit. Some of this appears to be the result of operating the lagoon at the extreme ends of the typical discharge during periods of high wet weather flows. City Public Works staff have made several adjustments to their operating procedures with an increased focus on the decanting level which is believed to have contributed to the improved effluent quality during the past two years.

The effluent BOD concentration levels have, with the exception of a few isolated high readings, been below the 30-mg/l limit. In the 2011-14 period average annual effluent BOD concentrations ranged from 12.2 to 28.2 mg/l. Table 3.6 summarizes the BOD and Solids % Removal between 2011– 2014.

Table 3.6 BOD and Solids Removal – 2011 to 2014

Month	Percent BOD_5 Removal (%)	Percent TSS Removal (%)	Month	Percent BOD_5 Removal (%)	Percent TSS Removal (%)
	2011			**2012**	
January	84.7	82.3	January	84.7	82.3
February	86.4	74.3	February	86.4	74.3
March	79.0	83.2	March	79.0	83.2
April	79.0	88.3	April	79.0	88.3
May	73.9	88.8	May	73.9	88.8
November	90.9	92.8	November	90.9	92.8
December	90.7	91.3	December	92.3	83.6
Average	**83.5**	**85.8**	**Average**	**85.5**	**79.3**
Maximum	**90.9**	**92.8**	**Maximum**	**92.3**	**89.2**
Minimum	**73.9**	**74.3**	**Minimum**	**77.2**	**49.7**

2013			2014		
January	97.5	92.4	January	93.1	85.5
February	94.0	87.8	February	73.7	78.6
March	87.2	87.8	March	84.0	84.7
April	80.5	89.8	April	90.9	71.4
May	87.0	83.2	May	91.2	95.3
November	92.8	91.9	November	88.6	85.3
December	-	-	December	85.3	89.6
Average	**89.8**	**88.8**	**Average**	**86.7**	**84.3**
Maximum	**97.5**	**92.4**	**Maximum**	**93.1**	**95.3**
Minimum	**80.5**	**83.2**	**Minimum**	**73.7**	**71.4**

3.2.8.2 Total Suspended Solids (TSS) Removal

Effluent Total Suspended Solids (TSS) – With the exception of February 2013, the lagoons have successfully maintained discharge concentrations of TSS below the permitted monthly average of 50 mg/l. When this month is excluded from the historic records during the time frame from 2011-14, monthly average effluent concentration levels ranging from 12.9 to 20.3 mg/l were recorded.

The City's NPDES permit has a requirement for 65-percent removal of TSS. Between January 2011 and December 2014 there were six measurements where the percent removal was below the 65-percent threshold. Four of the low readings occurred during periods of high I/I and are likely the result of very week influent concentrations during those months. Under the best of circumstances facultative lagoons have a theoretical limit to the effluent concentration they can produce. During these months the influent concentrations were unusually close to this theoretical limit. Despite this the concentration and the mass load of solids discharged to the river remained within allowed limits.

Despite the excursions noted above, the average percent removal between January 2011 and December 2014 ranged between 79.3 – 88.8 percent removal.

3.2.8.3 Bacteria Removal

E. Coli Bacteria – E. Coli bacteria levels are typically below the 406/100-ml limitation for a single sample. If this limit is exceeded, five consecutive re-samples may be taken at four-hour intervals beginning within 48 hours after the original sample was taken. If the log mean of the five samples is less than or equal to 126 organisms per 100 mL, a violation shall not be triggered. Occasionally, higher levels have been observed. The origin of these bacterial excursions is not known, since the residual chlorine levels were maintained for each month, but lagoon performance was not likely the reason. Slight changes in the chlorine concentration are believed to be possible to reverse this trend without exceeding allowed chlorine residuals.

3.2.8.4 Disinfection

Total Chlorine Residual - Residual chlorine levels in the effluent are typically somewhat elevated, but below the existing, permitted maximum of 1.0 mg/l. The 1.0-mg/l residual chlorine standard is high by present standards, but useful to the operator in achieving good bacterial kill when solids levels are elevated due to algae. Future permits may limit the effluent residual chlorine level to "non-detect," or virtually zero. It is very likely that de-chlorination or UV will be required as part of the next NPDES permit renewal.

3.2.8.5 Storage Capacity

The ability to store wastewater for 5 months is a prime asset of lagoons, and a capability most wastewater plants don't have. The original design indicates that the City lagoon area of 51 acres provides approximately 76 million gallons (MG) of storage if lagoons are drawn down to the minimum depth on May 31st. During the 5-month no-discharge season when storage is needed, the City's average dry season flow of 0.981 mgd is partially offset by evaporation and infiltration. As shown in Table 3.2 *Existing Treatment Facility Design Data*, Lagoons 1, 2, 3 and 4 were originally designed to provide usable depths of approximately 5, 4.6, 4 and 4.6 feet, respectively. These operating depths assumes that 2 feet of water remains over the bottom of the lagoon at minimum depth, and 2 feet of freeboard remains from the water surface to the top of the lagoons at maximum water level. This provided a dry weather holding storage of 76 MG under optimal operating conditions.

During an average year, approximately 3.4 feet of precipitation (56 million gallons) lands in the lagoons. This does not seriously impact the sewage storage budget, because most precipitation occurs during the 7-month discharge season when flow in the river is high. During the dry, no-discharge season, rainfall is low and evaporation is significant, both of which work to the system's advantage.

As noted in the following section, over time solids have accumulated in the bottom of the lagoon. As a result the storage capacity of the lagoons has been reduced. In August 2012 a field survey was completed to measure elevations at critical points to assist with the estimation of remaining storage volume. The field survey measurements included spot elevations at the top of the lagoon berms, at the bottom of the lagoon cells, and at the top of the sludge layer within each cell. It is important to note that the readings for the top of the solids and bottom of the lagoon cells were obtained at the wooden walkways for each cell and provide a discrete point of information but not a detailed topographic survey. This information was used in conjunction with the 2009 sludge survey to estimate the amount of storage volume that is no longer available.

Based on this data it is estimated that the total operating (storage) volume has been reduced to 52 MG if lagoons are drawn down to the minimum depth on May 31st. The assumed minimum depth allows for a nominal 2-foot cover over the layer of solids.

Similarly a minimum freeboard of 2-feet is assumed at maximum water height. This represents a reduction of approximately 32% from the original design volume and can only be recaptured through a sludge removal program.

3.2.8.6 Solids Management

Solids are settled from the wastewater to reside in the bottom of the lagoons. Influent Lagoon No. 1 receives most of the solids deposition, and the influent pipes and scour pads were relocated in 1998. A field survey of the lagoon solids was completed in Fall 2009 by the Oregon Association of Water Utilities. The study confirmed that due to accumulation of sludge the operating depth and consequently the summer storage volumes have been reduced. This was most pronounced by the presence of "islands" noted in Lagoon No. 1 in the vicinity of the original influent discharge, and one other significant deposition in southwest corner of Lagoon No. 1. Lagoon Nos. 3 and 4 had less solids accumulation than Lagoon Nos. 1 and 2.

When accumulated lagoon solids become excessive, contract removal of the sludge is needed if the lagoon is to be maintained and operated efficiently. If the selected alternative for future treatment requires restoring the design volume in any given cell, contract removal of sludge will have to be conducted. Contract removal costs can be significant and its main redeeming feature is that is conducted infrequently.

Because the sludge remains hidden below the water surface it is easy to forget the issues of solids management until operators begin to note sludge islands and sludge covered inlet pipes. Wastewater sludge management – whether for a lagoon system or a mechanical treatment plant – represents a significant part of the system's operation and maintenance cost over time. Based in the review of 2011 – 2014 DMR's it appears that sludge accumulation already has impacted the operation of lagoons during summer storage operations. A Biosolids Management Plan must be developed, approved and authorized by DEQ prior to beginning solids removal.

3.3 Existing Wastewater Treatment Needs

To date the existing wastewater treatment facility, including the 1978 expansion, has been capable of meeting the needs of the city for 37 years. This is almost twice as long as the anticipated design planning horizon. However, it is very close to the theoretical limits for key operational parameters. One key indicator is the decreasing effectiveness of BOD reduction which is trending towards permitted limits. Because in lagoon systems there is a time lag of many days between when influent enters the lagoon system and when it is discharged, the percent removal limit does not always trigger regulatory action. A second parameter is the mass load of BOD to the receiving stream and on average the system has been able to operate within the permitted limits. However, during the 2011-2014 timeframe, on four occasions the daily mass loading exceeded the 1000 lbs/day allowed under the permit. All events occurred during the months of October 2011 and 2013 when

DEQ granted the City's request to discharge early. During the remainder of the months and years the mass loadings have been well within the permit limitations with average readings below 400 lbs/day. Because most of the high readings occurred in October high algae growth that is visible on the lagoon surface is believed to have been a contributing factor.

A second key area is the ability of the system to successfully store all of the flows during the no-discharge summer holding period between the months of June through October. This has become increasingly difficult and early release was requested by the City and granted by DEQ every October between 2011 and 2014. A review of the staff gage readings at the lagoons provides insight to the volume of water stored during the required holding period of June through October. Using the staff gage readings during these years calculated stored volumes indicate a range of 48.6 to 53 MG prior to release.

In summary the City requires:

1. Additional biological treatment capacity year round.
2. Additional capacity to manage treated wastewater during the summer holding period.

To improve efficiency and safety a number of improvements are warranted. These are discussed in more detail in Section 5 of this report with associated cost estimates provided in Sections 6 and 7.

3.4 Infiltration and Inflow (I/I) Review

I/I is an on-going concern for most communities in the Pacific Northwest. The combination of high amounts of precipitation and relatively "tight soils", produce conditions that direct significant amounts of groundwater and surface runoff into sanitary sewers. Over time, the condition of mainlines, service laterals and manholes in any collection system will continue to deteriorate allowing more I/I into the system.

The collection system for the City of Independence is relatively old and I/I has been reported and documented to be significant, particularly in older segments of the collection system. System wide improvements reduced I/I as a result of the *1998 Sewer System Improvement Project*. The 1998 improvements were targeted at elimination of the most cost effective portion of I/I. The remaining contributing faults are believed to be wide spread throughout the system consisting of joints and faults that produce an accumulative amount of I/I that although significant, will require widespread and comprehensive rehabilitation. The level of effort anticipated to correct these remaining leaks is at a level that is not believed to be a cost effective approach to managing peak wet weather flows. Peak wet weather flows resulting from I/I remain in an order of magnitude of 10 times that of average dry weather flows. However, because major I/I sources were previously corrected the level of effort and associated capital costs anticipated to correct these

remaining leaks is at a level that is not believed to be a cost effective approach to managing peak wet weather flows.

Typically, I/I consists of relatively clean water that would otherwise not require treatment. However, once it is introduced to the sanitary sewer and mixes with domestic sewage the entire waste stream must be transported, treated and disinfected before discharging into a receiving stream or waterway. A large volume of I/I is undesirable for a number of reasons:

- I/I is relatively clean water that must nevertheless be transported and treated representing unnecessary energy and chemical consumption.
- In old poorly maintained collection systems, the magnitude of I/I is frequently of a magnitude of 5 to 10 times that associated with domestic sewage. This requires that collection and treatment facilities be oversized with capacity that is unused much of the time.
- Due to its relatively dilute strength, high levels of I/I will tend upset treatment processes in mechanical treatment facilities.
- I/I is an indication that the gravity mainlines, service laterals and appurtenances are leaking. During dry weather this represents a potential source of exfiltration and groundwater contamination.
- I/I uses hydraulic capacity of gravity sewers, pump stations and treatment facilities requiring frequent and expensive upsizing and replacement of key system components. This capacity would otherwise be available to accommodate growth.
- I/I will continue to become worse as sewerage system components age. Eventually the magnitude of the flows will exceed the hydraulic capacity of the collection system resulting in overflows and spills.
- Unidentified inflow sources may include local streams and waterways. Without specifically identifying the sources of inflow, communities continue to waste money building increasingly larger facilities without understanding that it will be virtually impossible to capture, transport and treat the peak flows. It is believed that these types of sources were identified and eliminated in 1998.

A great deal of I/I investigation and analysis was completed in preparation as part of the *1996 Sanitary Sewer Preliminary Design Report*. The 1996 report contains general strategies as well as specific recommendations for addressing I/I. In response to that analysis and report major improvements and modifications were constructed in the collection system removing the worst and most cost effective sources of I/I. The remaining sources are believed to be widely distributed throughout the collection system and will require an on-going maintenance program to resolve. For this reason focus of the current Sanitary Sewerage System Facilities Plan is to identify elements of the citywide approach to management of wastewater and sewerage systems.

Based on the analysis completed 1996, it is not believed to be cost effective to pursue a significant I/I reduction program versus constructing additional wastewater treatment. The existing lagoon cells are very effective at buffering peak flows during all seasons. The magnitude of I/I affecting the summer storage is very low so the City is better served committing capital funds to development of a new spray irrigation facility to address summer storage capacity. Similarly since I/I is primarily clean water, removing this from the influent flow stream will not recapture biological treatment. Consequently reducing I/I will have no impact on the need to increase biological treatment to address future population growth.

For efficiency, only key recommendations identified in the 1996 Preliminary Design Report are repeated. In general it should be noted that it is in the city's best long-term financial and regulatory interests to continue to address I/I throughout the system in an organized and comprehensive manner.

3.4.1 Previous Infiltration and Inflow Analysis

The impact of I/I on the City's collection system has been the subject of considerable investigation and a number of capital improvement projects. In 1977 and 1983, studies were completed of the sanitary sewer system by C&G Engineering and Westech Engineering respectively. The C&G study resulted in the expansion of the City's wastewater treatment facility and some collection system modifications. The Westech study was focused on I/I impacts and resulted in a limited pilot project calling for the replacement of some mainlines in Basin B. However, the approach was not comprehensive and consequently little appreciable impact to overall I/I was realized. In particular, overflows from manholes and pump station bypasses continued to occur during most rainfall events.

In 1994, in response to a Stipulation and Final Order (SFO) from Oregon Department of Environmental Quality (DEQ), a Sanitary Sewerage Facilities Plan Update was prepared by ASCG, Inc. This report focused on a system wide analysis to identify the most cost-effective approach to eliminating collection system overflows. Review comments from DEQ staff identified additional information that they felt should be obtained and analyzed before final design was initiated. The additional information focused primarily on identifying the specific locations of inflow sources within the collection system, as well as the magnitude of inflow contribution. Recommended improvements were also to be of a more specific nature than was provided for in the 1994 Facilities Plan Update.

In response to DEQ's evaluation and request for additional specificity, the City hired David Evans and Associates, Inc. to complete a *Sanitary Sewer Preliminary Design Report*. Work on the Preliminary Design Report began in 1994 and was completed in November 1996. The preparation and development of the Preliminary Design Report included a heavy emphasis on collecting field data during wet weather.

All previous investigative work on the Independence sewerage system indicated that the contributions of a few inflow sources far exceeded those from infiltration sources. Data analysis during the preparation of the 1996 Preliminary Design Report confirmed this assertion. Specific inflow sources identified included catch basins and roof drains in the downtown area, as well as a number of private service laterals throughout town. Infiltration appeared to be the result of widespread joint failures and similar collection system faults that would be expensive and difficult to correct.

In addition to the rise in influent levels attributed to inflow sources that collect surface runoff directly, it was noted that the influent measurements displayed a second more dramatic jump when the level of the Willamette River reached or exceeded the elevation of 141 feet. Subsequent topographic survey and field reconnaissance work identified a number of inverts in the lower sections of the Creek Interceptor and North Interceptor, which were at or below this elevation.

As a follow-up to the 1996 Preliminary Design Report, the City completed a $3.2 million (1998 dollars) Sewer System Improvement Project in 1998. The improvement project eliminated known major inflow sources including hydraulic connections to high water regions adjacent to Ash Creek and the Willamette River, replaced approximately 100 service laterals, lowered the hydraulic grade line and expanded the hydraulic capacity of several interceptors and pump stations. The project was successful at eliminating wet weather overflows and dramatically reducing overall peak system flows.

Although the Sanitary Sewer System Improvement project was very successful, the improvements targeted major system wide approaches and what were identified as the most cost effective rehabilitation/corrective work, but was not intended to and did not eliminate or correct all known deficiencies in the collection system. In addition to these improvements, the Preliminary Design Report (1996) identified an on-going maintenance and capital improvement program that would be required to address the system wide I/I concerns. These recommendations were incorporated into the 2008 Wastewater Master Plan.

As part of a follow-up on-going capital improvement program beginning in 1999, city Public Works staff completed several I/I investigation and sewer rehabilitation projects.

3.4.2 Infiltration and Inflow (I/I) Review

For the reasons noted above, many communities recognize that is more cost effective and environmentally responsible to address I/I as part of their on-going operation and maintenance programs. However, before undertaking such a program it is important to understand the nature of I/I and to recognize the elements that must be present for a successful reduction program.

Components of A Successful I/I Reduction Program

Some basic rules of thumb learned from successful by correction programs are:

Establish Baseline Flow Measurements: Measure wet weather I/I flows from the selected basins before repair work starts to establish a baseline and identify the areas contributing the highest amount of I/I. Establishing a baseline flow analysis begins with keeping complete and concise daily pump run records for each pump station and daily precipitation records. The next step is to complete wet weather-nighttime flow mapping at key manholes in the area. Flow mapping can be a valuable tool in determining where specific I/I is occurring.

Address inflow and Rain Induced Inflow (RII) First: Initial efforts should be directed at faults within the collection system, which are contributing significant amounts of I/I (usually inflow or RII sources). However, it is important to recognize that not all of the major contributors will be located and corrected during the first step. Some will not be discovered until initial repairs are made. I/I is the result of the interaction of several factors that are difficult to control. This includes the amount of precipitation, intensity of precipitation, groundwater conditions, levels of adjacent waterways, and the time interval between storms. Surcharging of sewers may prevent some I/I from entering the collection system until sufficient improvements have been made to remove the surcharge. With the exceptions of the South Interceptor there are no reported interceptors consistently surcharging within the collection system.

Comprehensive Program: If large amounts of infiltration and collection system faults are present, corrective work must be completed in a comprehensive manner within the entire sub-basin. It is a waste of money to repair only the large cracks in the main line and ignore the smaller faults, and holes and service laterals. Significant reductions in infiltration are only reliably achieved by eliminating all or nearly all of the infiltration contributing faults within a sewer basin or sub-basin. A lesser repair tends to shift the entry point of I/I from one location to the next.

Budget: Establish an on-going inspection and repair plan and budget including allowances for the identification and repair of faults not initially discovered. The repair plan must include repairs to mainlines, manholes and service laterals if each contribute significant I/I. Recognize that without an ongoing I/I maintenance and repair program, I/I will only get worse. Old main lines, manholes and service

laterals will continue to deteriorate with time, creating new problems and making existing ones worse.

Construction Inspection and Testing: A vigilant inspection and testing program for new construction must be established and enforced. Requiring high quality construction materials, adequate trench preparation and backfill, and pressure testing of mainline pipes, service laterals, and manholes, will ensure that new construction does not make the City's I/I problems worse. If properly constructed, modern construction materials for sanitary sewers can produce collection systems with extremely low initial levels of I/I. The City has been diligent implementing this since 1998.

The City has had an ongoing proactive construction inspection and testing program for approximately 17 years. Continuing this program will reduce the potential of high I/I flows.

Iterative Process: Successful I/I rehabilitation is an iterative process where the City should make the initial repairs then re-measure wet weather I/I. Unless the I/I reductions are acceptable, the remaining sources should be located and repaired. This process should be repeated until acceptable I/I reductions are achieved. Removing sources of inflow and RII usually produces the most cost-effective reduction in I/I.

3.5 Existing Flow Conditions

In analyzing and planning for sanitary sewer systems two general flow conditions must be considered: system wide flows, and flows within a basin. System wide flows include the sewage from the entire collection system that is pumped by the influent pump stations into the headworks located at the WWTF. The system wide flows are measured at the Influent Flow Meter where a circular chart recorder tracks the influent flows on a continuous basis. The chart recorder shows the various influent pump stations starting and stopping their pumps, and the increase in pumping frequency as sewage flow fluctuates through the 24-hour day, and during and after storm events.

Basin wide flows are often apparent at each influent pump station where the time of each pump on and pump off are recorded. Each influent pump station collects the sewage from the basin via a network of gravity sewers and lift stations. Sewage collected at one of five major pump stations is then pumped directly to the WWTF headworks. Each basin has boundaries that have been established based on geographic and topographic constraints, or residential development.

System wide flows are used to size treatment facilities and plan for potential permit renewal impacts. In addition, system wide flows aid in determining near term improvements to the existing WWTF improvements and identifying long term future WWTF improvements.

Basin flows are used to size gravity interceptors, pump or lift stations, and gravity collection system pipes. Basin flows aid in determining existing and future collection system deficiencies and future collection system improvements.

This Section describes both the existing system and basin flows for the sewage collection system.

3.5.1 Historic and System Wide Flows

Historic BOD, TSS, influent, and precipitation are summarized in the City's DMR's. Sewage flows are measured daily, in terms of million gallons per day (mgd).

Because the magnitude of sewage flow is always changing, it is defined with respect to categories for the frequency of occurrence for the flow; e.g., average, maximum, and peak. These categories, the "flow conditions," are defined in the DEQ permit and design guidelines.

3.5.1.1 Definitions for Wastewater Flows and Parameters

There are a number of flow conditions that must be considered in the evaluation, planning and design of wastewater and sewerage systems. The Oregon Department of Environmental Quality (DEQ) has established "*Guidelines for Making Wet-Weather and Peak Flow Projection for Sewage Treatment in Western Oregon"* which is a technical guideline document. The guideline outlines specific wet-weather and peak flow conditions and recommended procedures for developing these estimated or projected flow conditions. The following paragraphs summarize the DEQ guidelines. The calculations and graphs used to complete these flow estimates are included in Appendix B of this report.

Average Dry Weather Flow (ADWF)

Average Dry Weather Flow is defined as the average of daily flows over the 6-month dry-weather period, roughly May through October. This is the flow-rate on which dry-weather mass loads are based for wastewater treatment facility design and evaluation. It can also provide a check for low hydraulic flow conditions in collection system components.

Average daily flows are measured and recorded daily in the Discharge Monitoring Records (DMR's). In preparing this report DMR's for the period from January 2011 through December 2014 were reviewed to identify an ADWF representative of the current conditions. The City's ADWF during this period was 0.981 mgd.

Peak Average Daily Flow ($PADF_5$)

Peak average daily flow is the peak daily flow resulting from a 5-year 24-hour storm. DEQ guidelines recommend developing $PADF_5$ from plant records and rainfall data. The $PADF_5$ is an important consideration when designing collection

system pumps and the wastewater treatment facility. For Independence the 5-year, 24-hour storm assumes 3.5 inches of precipitation and is estimated to be 8.3 mgd.

Maximum Monthly Dry Weather Flow ($MMDWF_{10}$)

The Maximum Monthly Average Dry-Weather Flow with a 10-percent probability of occurrence is designated $MMDWF_{10}$. It is assumed to be the monthly average flow in the rainiest summer month of high groundwater. West of the Cascades $MMDWF_{10}$ invariably occurs during May. In Independence the $MMDWF_{10}$ between is estimated to be 1.42 mgd using DEQ's graphic method.

Maximum Monthly Wet Weather Flow ($MMWWF_5$)

The Maximum Monthly Average Wet-Weather Flow represents the highest monthly average attained during the winter period of high groundwater. West of the Cascades, high groundwater is usually not attained until January, and the $MMWWF_5$ occurs in January. The 5-year January accumulation is listed as the 80% value or the amount of rainfall that exceeds 4 out of 5 totals that have been recorded in January. Using the DEQ graphic method in Independence the $MMWWF_5$ is estimated to be 2.21 mgd.

Wet Weather Peak Wet Hydraulic Flow ($WWPIF_5$)

PIF_5 is the peak instantaneous or peak hourly flow associated with a 5-Year PDAF. It can otherwise be described as the peak flow resulting from a 5-year storm during high groundwater periods. DEQ guidelines recommend developing the PIF_5 from plant records, or by estimation using observed diurnal peaking factors. PIF_5 is an important consideration when designing key hydraulic components in the collection system and influent handling facilities at the wastewater treatment facility. For Independence the 5-year, 24-hour storm assumes 3.5 inches of precipitation.

The PIF_5 combines domestic flows and infiltration and inflow (I/I). Similar to many communities with older collection systems, I/I can raise wet weather flows dramatically above those associated with dry months. Prior to the 1998 improvement project, portions of the gravity collection system were directly or indirectly connected hydraulically to the Ash Creek and the Willamette River flood plains and inflow was extreme during periods of high water. As a result of the system modifications and improvements completed in 1998, the known inflow sources from flood plains have been eliminated and inflow has been reduced dramatically.

A continuously recording "circle chart" has been in operation since 1998 and provides a measurement and record of the peak instantaneous flows delivered to the WWTF. However, it is important to note that the influent flow received at the

WWTF is discharged from one or multiple pump stations. For this reason the peak flow measured at the Parshall flume is inherently limited by the hydraulic capacity of the pump stations pumping simultaneously. It is also important to consider the impacts of the peak instantaneous flows immediately upstream from the key pump stations. The magnitude of instantaneous flows within a basin or interceptor is more difficult to quantify since it requires estimation without the benefit of direct measurement and recording. Obtaining daily pump run records for each pump station, completing periodic draw down tests to confirm the pumping rates, and keeping daily precipitation records will aid in predicting the peak instantaneous flows upstream of the influent pump stations. Flow mapping portions of each basin and immediately upstream of the influent pump stations sometimes provides a additional insight for predicting peak instantaneous flows, especially the influence of I/I and can be completed as part of an I/I investigation or Preliminary Design Report if it is deemed to be a critical component of the required capital improvements.

The City's $WWPIF_5$ (PIF_5) at the lagoons is estimated to be 10.7 mgd.

Biochemical Oxygen Demand (BOD) and Total Suspended Solids (TSS)

BOD is a measure of wastewater strength in terms of the quantity of oxygen required for biological oxidation of the organic matter contained in the wastewater. The BOD loading imposed on a treatment facility influences both the type and degree of treatment that must be provided to produce the required effluent quality. All references to BOD in this report are to five-day BOD at 20 degrees Celsius.

TSS is a measure of the total quantity of suspended material present in the wastewater. The quantity of TSS present influences the sizing of settling units, sludge handling, and disposal processes, as well as the effectiveness of disinfection.

Per capita BOD and TSS rates were established based on historic Discharge Monitoring Report (DMR's) for the period between 2011 and 2014. For BOD between 2011 and 2014 the per capita rates reported and calculated increased to the range of 0.192 – 0.228 lbs/capita/day. For planning purposes of projecting future load increases it is recommended that this be averaged to 0.212 lbs/capita/day.

TSS loadings had a range of 0.174 – 0.225. The average TSS loading during the 2011-2014 period is 0.195 lbs/day/person and will be applied for purposes of projecting future loading. These rates are within the expected range for similar municipalities and it is reasonable to apply this to the projected population growth when estimating future BOD and TSS loads to the WWTF.

Average BOD Loadings 2011 – 2014

Table 3.7 Average Annual BOD Loadings

Year	Average Annual Influent Concentration (mg/l)	Average Annual Influent Loading (lbs/day)	Average Annual Influent Loading (lbs/capita/day)
2011	198	1,852	0.215
2012	172	1,643	0.192
2013	226	1,804	0.211
2014	235	1,958	0.228
Average 2011-2014	***208***	**1,814**	***0.212***

As noted in Table 3.7 historic BOD loads on the primary lagoons (Nos. 1 and 2) produce a load of 1,814 lbs/day or 64.8 lbs/day/acre which is above the 50 lbs/acre/day maximum loading rate assumed for primary lagoons in Oregon communities west of the Cascades. Operational modifications allow the use of cell No. 3 as additional primary lagoon surface area and when this is taken into account the total areal loading for lagoon Nos 1, 2 and 3 is 50.3 lbs/acre/day. The overall loading rate for the entire lagoon system is 35.5 lbs/day, which is at the 35 lbs/day limit for total treatment surface area.

Applying these broad design guidelines confirms that the treatment facility appears to be at or very close to its maximum operational capacity. Assuming that lagoon cell Nos. 1, 2 and 3 can efficiently operate in parallel as primary treatment cells, and rely only on cell No. 4 for final polishing would be the most optimistic scenario and is not advisable as a long term solution. Even under this optimum circumstance applying the design guidelines noted above the City should begin planning for construction of additional treatment capacity. As was noted in Section 3.2.8 monitoring records of the treatment performance and efficiency appear to indicate that on occasion the biological system is operating under stress which underscores the need to expand treatment.

3.5.1.2 Summary of System Wide Flows and Loads

Based on the analysis of the existing Discharge Monitoring Reports (DMR's), DEQ Guidelines, and current and historic population records, existing system wide flow and loads were determined. Table 3.8 below, summarizes the current flow loads for the City's WWTF.

Table 3.8 Summary of Current System Wide Flows/Loading

Flow Type	Current (2011 - 2014)		
	Units	Value	Per Capita Rate
Average Dry Weather Flow (ADWF)	mgd	0.981	N.A.
Peak Daily Average Flows ($PDAF_5$)	mgd	8.30	N.A.
Max. Monthly Dry Weather Flow ($MMDWF_{10}$)	mgd	1.42	N.A.
Max. Monthly Wet Weather Flow ($MMWWF_5$)	mgd	2.21	N.A.
Wet Weather Peak Hydraulic Flow ($WWPIF_5$)	mgd	10.7	N.A.
Total Suspended Solids (TSS)	lbs/day	1,671	0.195 lb/day/pc
Biochemical Oxygen Demand (BOD_5)	lbs/day	1,814	0.212 lb/day/pc

(1) BOD and TSS are averages based on 2011-2014 DMR's

3.5.2 Existing Basin Flows

Flow within each individual basin is estimated to assess the ability of the corresponding influent pump station and force main to meet the existing demands, and the effectiveness of I/I remediation within the basins. The first step in estimating the basin flows is to identify the existing and proposed land use within the basin, and apply the appropriate flow and loading factors. The information provided in the following sections should be viewed as general and more detailed analysis is required and warranted as part of a preliminary and final prior to sizing and constructing replacement of major hydraulic components in the collection system.

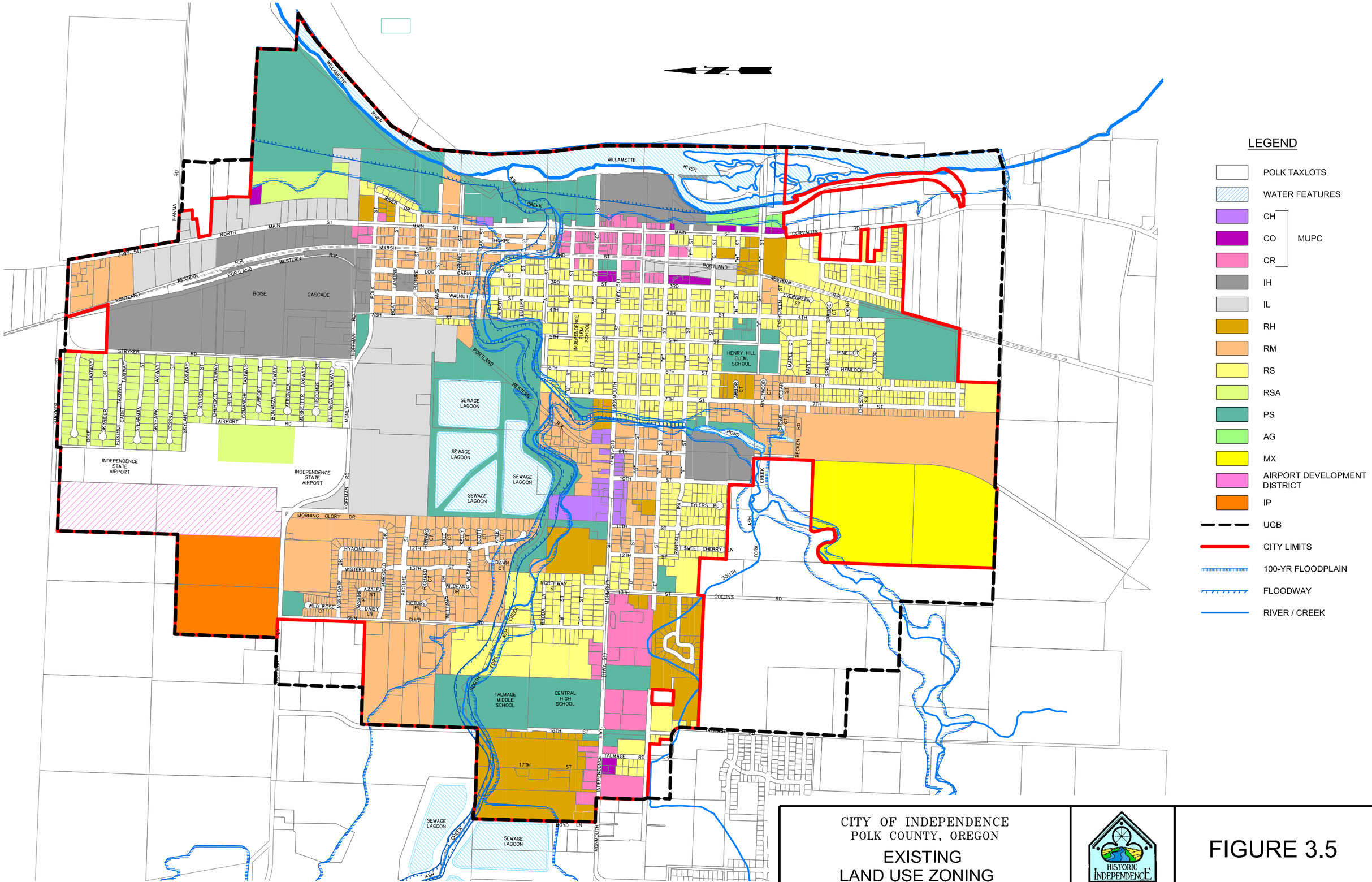
3.5.2.1 Existing Land Use Conditions

Land use within the City is diverse and includes the full spectrum of land use zoning including residential, commercial, industrial and public institutional. Table 3.9 provides a summary of land uses within the current City Limits identifying both developed and undeveloped properties. The total area within the City Limits is further broken down into areas tributary to each of the five main sewer basins. Potential expansion areas within the UBG will be discussed in more detail in Section 4 of this report.

The areas within each basin were measured directly from GIS information provided by the City of Independence and Polk County as part of the 2006 Wastewater Master Plan and updated with information provided by city staff reflecting annexations to the city limits and expansions to the UGB adopted in the past eight years. Table 3.9 also provides a summary of the areas and Figure 3.5 identifies the existing land use within the current UGB.

Table 3.9 Existing Land Use – City Limits

Basin	Influent Pump Station	Sub Basins	Total Developed Area (AC)	(RS / MX) Single Family Residential Dev.	(RM) Medium Density Residential Dev.	(RH) High Density Residential All Dev.	MUPC (Formerly CH/CO/CR) Commercial Dev.	(IH / IL) Industrial Heavy Dev.	(PS) Public Service All Dev.	R/W and Other Non Develop able N/A
				Land Use Type						
A	**Oak St.**	A	229.8	5.0	48.1	3.5	1.5	102.7	9.3	59.6
B	**Riverview**	B1	98.1	45.5	15.7	4.7	1.1	0.0	11.9	19.2
		B2A	114.2	51.0	0.2	0.0	13.4	9.2	3.6	36.8
		B2B	126.7	57.7	2.1	0.0	1.7	4.7	23.4	37.1
		Total	*339*	*154.2*	*18.0*	*4.7*	*16.2*	*13.9*	*38.9*	*93.1*
C	**9th St.**	C1	172.9	33.7	37.5	26.5	30.7	8.5	10.5	25.5
		C2	28.2	11.2	6.5	0.0	0.0	0.0	2.6	7.9
		Total	*201.1*	*44.9*	*44.0*	*26.5*	*30.7*	*8.5*	*13.1*	*33.4*
D	**Lagoon**	D1	101.8	9.3	0.8	26.0	13.2	0.0	33.5	19.0
		D2	128.9	0.0	91.8	0.0	0.0	0.0	0.0	37.1
		Total	*230.7*	*9.3*	*92.6*	*26.0*	*13.2*	*0.0*	*33.5*	*56.1*
E	**Williams**	E	129.4	66.0	0.0	0.0	0.0	24.4	0.0	39.0
		Grand Total	**1130**	**279**	**203**	**61**	**62**	**150**	**94.8**	**281.2**



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

EXISTING
LAND USE ZONING

FIGURE 3.5

3.5.2.2 Summary of Existing Basin Flows

Densities for each land use designation were estimated as part of the basin flow analysis. The estimated flows are based on land use, acreage, and existing per capita assumptions. Although this is not a precise analysis, applying the underlying land use zoning for areas contained in the basins provides a rational approach that takes into account the typical flow and loadings associated with these types of developments. Table 3.10 below summarizes the parameters used in estimating each basin flow.

Table 3.10 Parameters for Existing Basin Flow Estimates

Design Criteria	Existing Criteria
Number of Dwelling Units (DU) Per RS Acre	5
Number of Dwelling Unit per RM Acre	6
Number of Dwelling Unit per RH Acre	16
Number of Dwelling Unit per RSA Acre	2.5
Number of Dwelling Unit per MX Acre	5
Persons Per RS, RM, and MX Dwelling Unit	2.9
Persons per RH Dwelling Unit	2.5
Persons per RSA Dwelling Unit	2.0
Commercial Flow (gpad)	1000
Industrial Flow (gpapd)	1000
Gallons per Capita per Day (gpcpd)	75
Infiltration and Inflow (I/I) (gpad)	1000
Peaking Factor	3.0

Based on the above parameters the existing flow and peak flow for each basin was estimated and summarized in Table 3.11 below.

Table 3.11 Summary of Existing Basin Flows

Basin	Influent Pump Station	Total Existing Flow	Total Existing Peak Flow	
		(gpd)	(mgd)	(gpm)
A	Oak Street	412,860	0.779	541
B	Riverview	574,361	1.05	726
C	9Th Street	426,049	0.876	608
D	Lagoon	452,897	0.897	623
E	Williams St.	225,575	0.418	290
	Totals	**2,091,741**	**4.01**	**2,788**

3.5.2.2.1 Existing Collection System and Interceptor Capacities

The existing flow estimates for each basin were compared with each existing interceptor and pump station capacity. The interceptor capacity was determined from existing record information provided by the City. Table 3.12 below summarizes the existing interceptor and pump station capacities.

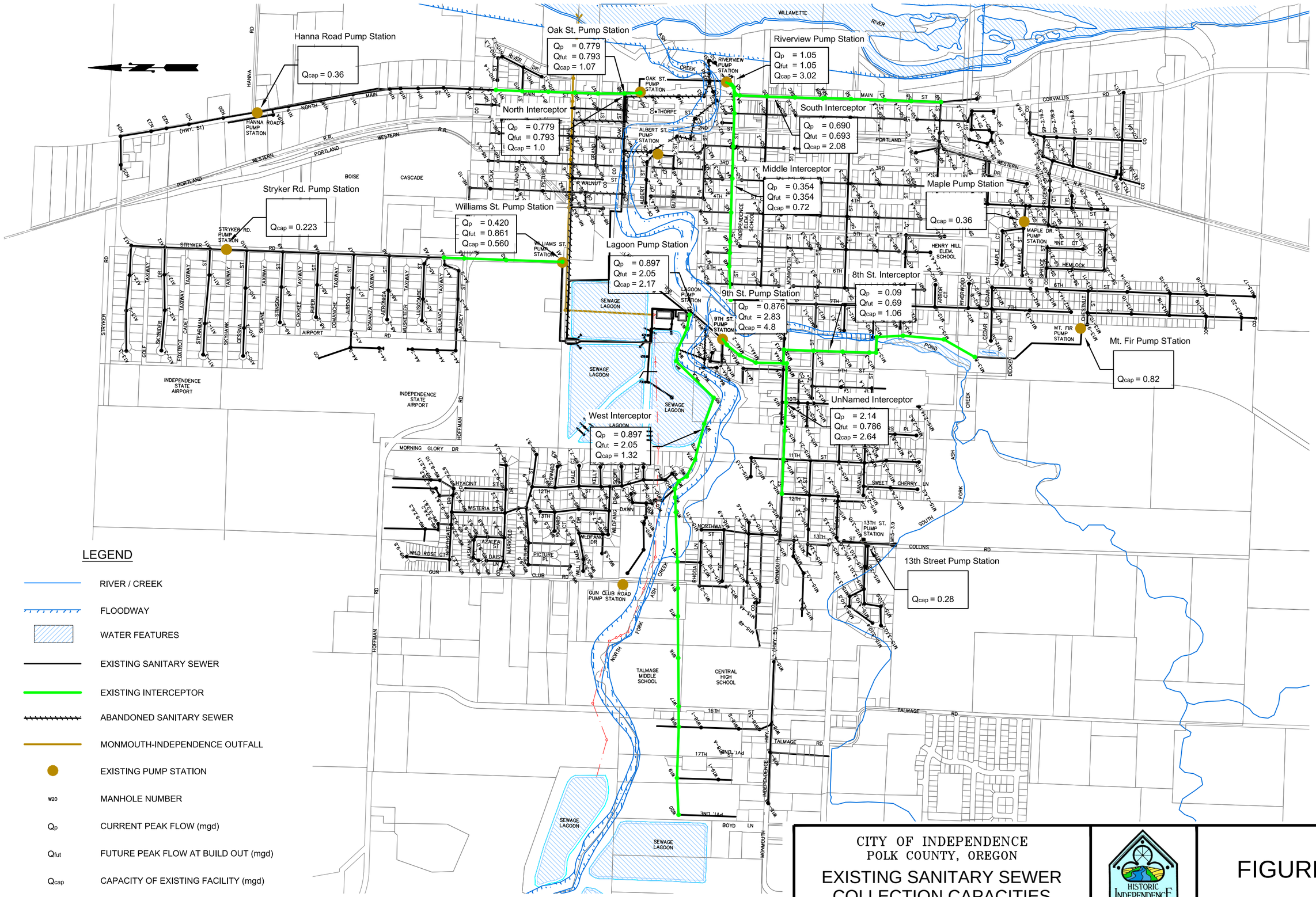
Table 3.12 Existing Interceptor and Pump Station Capacity

Basin	Influent Pump Station	Interceptor Name	Manhole Location	Interceptor Capacity (mgd)	Existing Peak Flow (mgd)	Qp/ Qc	Existing Pump Station Flow (mgd)	(gpm)
A	Oak St.	North	N6	1.26	0.779	0.62	1.06	740
			N11	1.39(3)			0.36	250
B	Riverview	Confluence	M1	6.1	1.05	0.17	3.00	2080
		Middle	M2	0.72	0.354	0.49		
		South	S4	2.08	0.69	0.33		
C	9th St.	Confluence	M14A	4.80	0.876	0.18	1.30	900
		Monmouth	M14B	2.64	0.786	0.30		
		8th Street	M13-4	1.06	0.09	0.08	0.82[1] 1.00[2]	568 700
D	Lagoon	West	W5	1.32	0.897	0.68	2.17	1509
E	Williams	Airpark	A2	0.56	0.42	0.75	0.36	246

[1] Mt. Fir Pump Station 1 pump running
[2] Mt. Fir Pump Station 2 pumps running (room for 3 pumps)
[3] Sanitary Sewer Improvements; OR Hwy 51 Stryker Road to Polk Street.
[4] New Hanna Road Pump Station

Projected growth will require improvements and/or replacement of many of the key facilities and will be discussed in more detail in Chapter 4. Alternatives for addressing projected capacity shortfalls will also be identified and developed in more detail as part of Chapters 4 and 5.

Figure 3.6 includes the existing and projected flows and capacities of the pump station and interceptors. The projected flow analysis is described in detail in Section 4 and is included in this figure for clarity.



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

EXISTING SANITARY SEWER COLLECTION CAPACITIES

HISTORIC INDEPENDENCE

FIGURE 3.6

4 FUTURE CONDITIONS

When projecting flows within the sewerage system, it is important to apply two different strategies. First is the projection of total system wide flows and loads to the WWTF, and second is projecting flows within specific basins and tributary to main components within the collection system. This requires consideration of the historic and projected population growth, and the specific land uses allowed and anticipated for land within the city limits and Urban Growth Boundary (UGB).

4.1 Population Growth and Land Use

Table 4.1 summarizes the population history for the City between the years 1930 and 2015. Independence has experienced fluctuating periods of rapid and relatively flat growth. Growth from 1980 through 1990 was relatively slow including years with almost no population increase. This corresponded with the decline of timber industry and the discontinuation of several local industrial operations. Shortly after the 1994 Facilities Plan Update, the City began to experience a dramatic increase in population growth including several residential developments. Between 1995 and 2008, the total population increased by 3,155 persons which represented a population increase of 64-percent. Annualized this represents an average yearly population increase of 4.0-percent. A slowdown in the national and local economies between 2008 and 2015 is reflected in a reduced growth rate (0.6% annualized). Recently the City has reported an increase in private development applications and construction and anticipates a return to a population growth. The Portland State University Population Research Center reported the 2013 population as 8,585. The City estimates the 2015 population to be 8,605.

Overestimating the rate of population growth could result in significant capital cost in facilities that are oversized and never fully utilized before reaching the end of a useful design life. The opposite problem would be under estimating the size of the flows and loads and constructing a facility with insufficient capacity to accommodate population and industry growth. A review of the historic population trends between 1930 and 2015 indicates that the average annual growth rate ranged between 0 and 4%. To strike a balance between the high and low cycles a projected population growth of 2.8% has been used in developing land use planning documents for Polk County, and has been incorporated into the development of this plan. Applying this growth rate to the current population of 8,605 will result in a projected population of 14,949 for the year 2035.

The projected citywide population will be used in conjunction with historic performance and standard industry design parameters to project flows and loads to the WWTF.

4.1.1 Historic and Projected Population Growth

Table 4.1 Historic and Projected Populations

Year	Population	Year	Population
1930	1,250	**1985**	4,225
1935	1,325	**1990**	4,425
1940	1,400	**1995**	4,875
1945	1,700	**2000**	6,035
1950	2,000	**2003**	6,850
1955	2,000	**2004**	7,170
1960	2,000	**2015**	8,605
1965	2,250	**2020**	9,879
1970	2,500	**2025**	11,342
1975	3,262	**2030**	13,021
1980	4,024	**2035**	14,949

4.1.2 Future Land Use Conditions

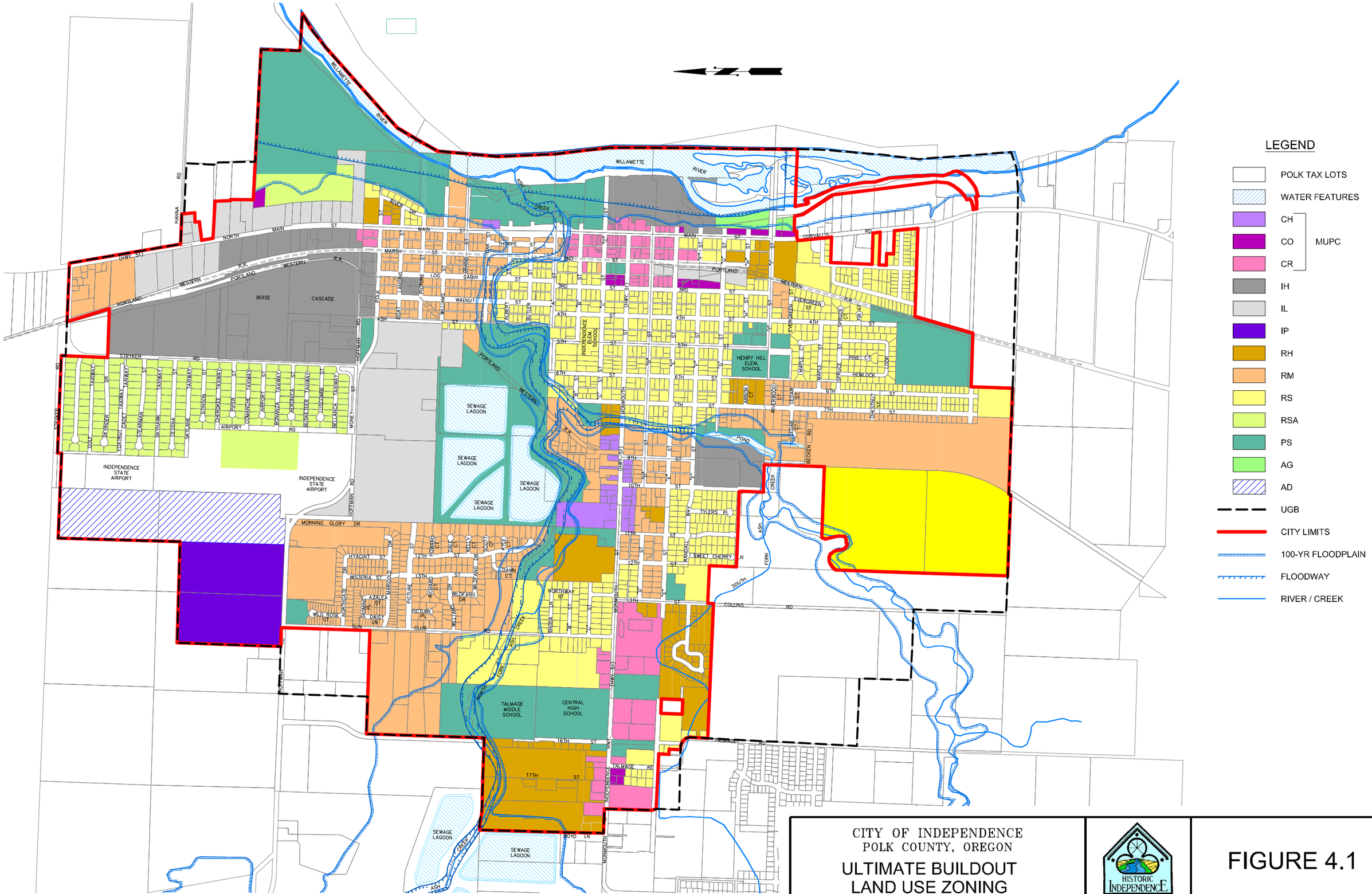
The potential change or continuation of land use outside of the existing City Limits and within the existing Urban Growth Boundary (UGB) has not been established at this time. Establishing future zoning was determined primarily by extending or continuation of the existing land use within the UBG limits and for estimating purposes in this report it has been assumed to be MX land use equivalent to single family residential development.

The areas within each basin were measured directly from GIS information provided by the City of Independence and Polk County and updated with information provided by city staff regarding expansion of the city limits and UGB adopted during the past eight years. Table 4.2 summarizes the land use areas within the UBG, including assumed UGB expansions.

Land use zoning designations for properties within the existing UGB are shown in Figure 4.1.

Table 4.2 Land Use at Ultimate Buildout

						Land Use Type											
						(RS / MX) Single Family Residential		(RM) Medium Density Residential		(RH) High Density Residential		(MUPC) Formerly CH/CO/CR Commercial		(IH / IL) Industrial Heavy		(PS) Public Service	R/W and Other Non Develo pable
Basin	**Influent Pump Station**	**Sub Basins**	**Total Area (AC)**	**Total Developed Area (AC)**	**Total Un-Developed Area (AC)**	**Dev.**	**Un Dev.**	**Dev.**	**Un Dev.**	**All Dev.**	**Un Dev.**	**Dev.**	**Un Dev.**	**Dev.**	**Un Dev.**	**All Dev.**	**N.A**
A	**Oak St.**	A	233.3	229.7	3.6	5.0	1.0	48.1	0.0	3.5	0.0	1.5	0.0	102.7	2.6	9.3	59.6
B	**Riverview**	B1	98.1	98.1	0.0	45.5	0.0	15.7	0.0	4.7	0.0	1.1	0.0	0.0	0.0	11.9	19.2
		B2A	114.9	114.2	0.7	51.0	0.0	0.2	0.0	0.0	0.0	13.4	0.7	9.2	0.0	3.6	36.8
		B2B	126.7	126.7	0.0	57.7	0.0	2.1	0.0	0.0	0.0	1.7	0.0	4.7	0.0	23.4	37.1
		Total	*339.7*	*339*	*0.7*	*154.2*	*0.0*	*18.0*	*0.0*	*4.7*	*0.0*	*16.2*	*0.7*	*13.9*	*0.0*	*38.9*	*93.1*
C	**9th St.**	C1	476.8	172.9	303.9	33.7	279.6	37.5	2.4	26.5	11.1	30.7	5.6	8.5	5.2	10.5	25.5
		C2	161.5	28.2	133.3	11.2	84.5	6.5	48.8	0.0	0.0	0.0	0.0	0.0	0.0	2.6	7.9
		Total	*638.3*	*201.1*	*437.2*	*44.9*	*364.1*	*44.0*	*51.2*	*26.5*	*11.1*	*30.7*	*5.6*	*8.5*	*5.2*	*13.1*	*33.4*
D	**Lagoon**	D1	237.1	101.8	135.3	9.3	21.7	0.8	0.0	26.0	13.6	13.2	0.0	0.0	100	33.5	19.0
		D2	249.5	128.9	120.6	0.0	20.0	91.8	50.6	0.0	0.0	0.0	0.0	0.0	50	0	37.1
		Total	*486.6*	*230.7*	*255.9*	*9.3*	*41.7*	*92.6*	*50.6*	*26.0*	*13.6*	*13.2*	*0.0*	*0.0*	*150*	*33.5*	*56.1*
E	**Williams**	E1	238.7	129.4	109.3	66.0	22.6	0.0	0.0	0.0	0.0	0.0	0.0	24.4	86.7	0	39.0
	Grand Total		**1937**	**1130**	**806.7**	**279**	**429**	**203**	**102**	**61**	**25**	**62**	**6.3**	**150**	**245**	**94.8**	**281.2**



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

ULTIMATE BUILDOUT
LAND USE ZONING

FIGURE 4.1

4.1.3 Projected System Wide Flows and Loading

In projecting flows and loads delivered to the WWTF, the historic data related to these two design parameters were considered in combination with the projected population growth for the City as a whole. By analyzing the existing flows and treatment plant loadings, per capita unit parameters for flows and loadings can be established and applied to the projected population growth. Table 4.3 lists the parameters utilized for projecting flows and loads system-wide.

An important underlying assumption used in these projections is that the population, commercial services, and industries, which currently live in and serve Independence, will increase proportionally and be of a similar nature in terms the their associated flows and waste strength. If this is not true, and new industries or commercial enterprises with heavy flows and loads develop, these developments will need to be evaluated on a case by case basis to determine if pre-treatment or other methods are appropriate.

Table 4.3 Flows and Pollutant Load Factors

Per Capita Domestic Sewage	75 gpcpd
Infiltration Contribution from New Construction	1000 gpd/acre
Diurnal Peaking Factor	3.0
Existing Inflow Correlation	2.1 mgd/inch of Precipitation
Per Capita BOD Loading	0.212 lbs/day/capita
Per Capita TSS Loading	0.195 lbs/day/capita
Design Storm, 5-Year Recurrence	3.5 inches/day
Average Daily Infiltration	0.20 mgd

Based on the parameters listed in Table 4.3 and the population projections listed in Table 4.1 projected flow and loading values were completed. Table 4.4 summarizes the existing and projected value. Table 1.7 in the Executive Summary of this report include additional current and projected design flows that are important to sizing specific elements of new treatment facilities. The DEQ graphical methodology was used to estimate current design flows and is contained in Appendix B. Using the current design flows, the parameters listed in Table 4.3, and the population projections, future flow and loading values were estimated. Table 4.4 summarizes existing and projected values.

Table 4.4 Summary of Current and Projected System Wide Flows/Loading

Flow Type	Units	2015 Value	2035 Value
Influent Flow			
Average Dry Weather Flow (ADWF)	mgd	0.981	1.71
Peak Instantaneous Wet Weather Flow (PIF_5)	mgd	10.7	12.4
Influent Load			
Total Suspended Solids (TSS)	lb/day	1,671	2,915
Biochemical Oxygen Demand (BOD_5)	lb/day	1,814	3,169
Total Lagoon Loading Rate (Cells 1, 2, 3 & 4)	lb BOD/acre/day	36	N.A.
Primary Lagoon Loading Rate (Cells 1, 2 & 3)	lb BOD/acre/day	50	N.A.

4.2 Projected Basin Flows

4.2.1 Projected Basin Flows

While historic projections are good for projecting growth on a City-wide basis, it is not reasonable to assume that all of the currently undeveloped land within the City limits will be fully developed within the 20-year study period. Population growth will likely be concentrated in specific basins and most of the areas scheduled for development will be only partially developed to their full density. However, when sizing new pump stations and interceptor sewers, it does not make sense to construct them to accommodate only the partial development within the study period. Rather, these key facilities must be sized to accommodate the ultimate build-out through either full constructed capacity or phasing, even though full development may occur outside of the 20-year study period. For this reason, the flow projections for each future basin were completed using parameters listed in Table 4.5 below.

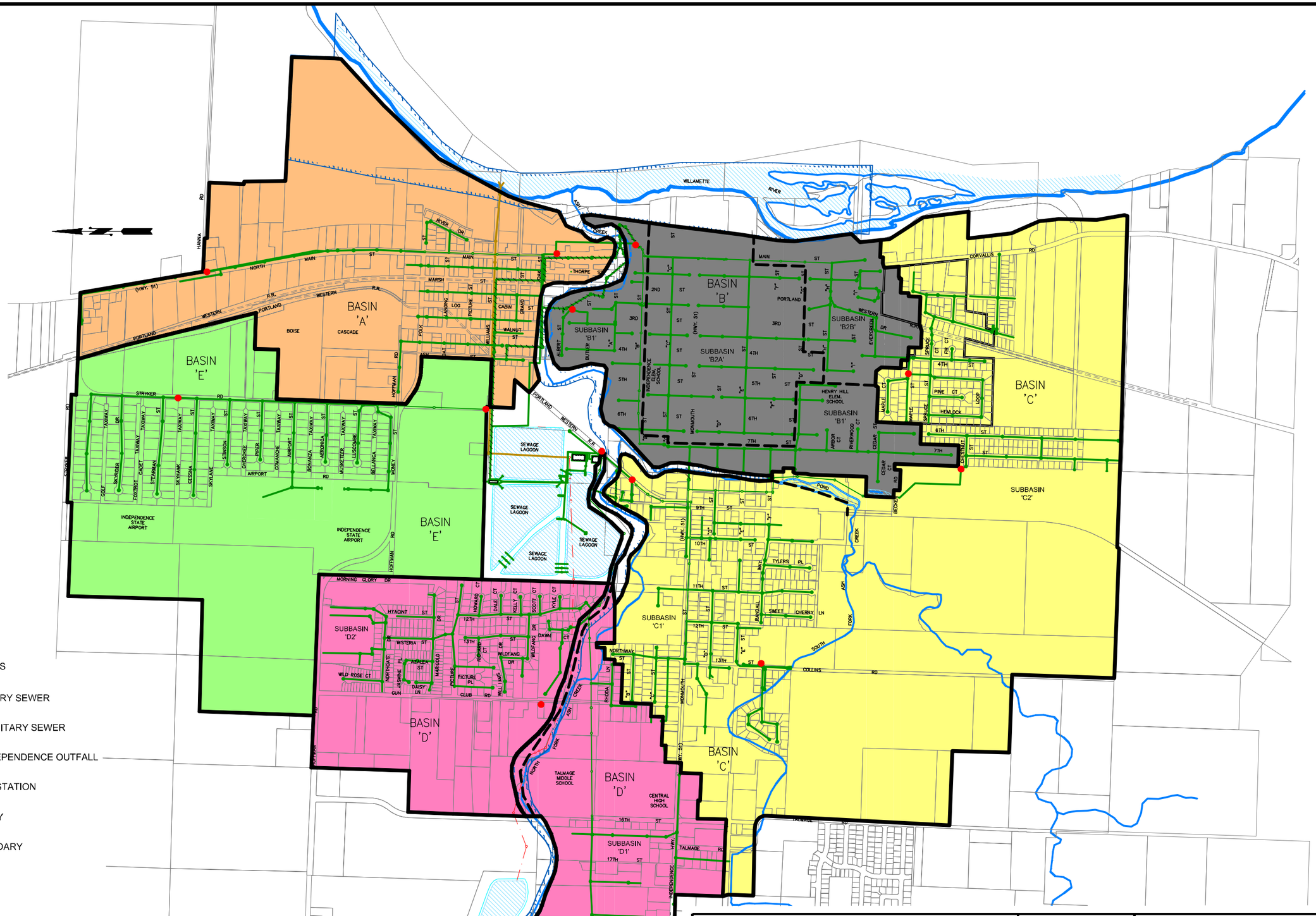
Table 4.5 Parameters for Projected Ultimate Flows within Basin

Description	Design Criteria
Number of Dwelling Units (DU) Per RS and MX Acre	5
Number of Dwelling Unit per RM Acre	6
Number of Dwelling Unit per RH Acre	16
Number of Dwelling Unit per RSA Acre	2.5
Persons Per RS, RM and MX Dwelling Unit	2.9
Persons per RH Dwelling Unit	2.5
Persons per RSA Dwelling Unit	2.0
Commercial Flow (gpad)	1000
Industrial Flow (gpad)	1000
Gallons per Capita per Day (gpcpd)	75
Infiltration and Inflow (I/I) (gpad)	1000
Peaking Factor	3.0

The land within each basin that is currently undeveloped has been summarized in Table 4.2. Unit flow and development density design parameters summarized in Table 4.5 are applied to these undeveloped areas to estimate projected flow increases. The increased flow is combined with existing flow estimates to produce the projected total flow for each basin at buildout. This information is summarized in Table 4.6 below. The configuration and limits of the proposed future basins are outlined in Figure 4.2.

Table 4.6 Future Basin Flow Projections

Basin	Influent Pump Station	Future Base Flow	Future Peak Flow	
		(mgd)	(mgd)	(gpm)
A	Oak Street	0.420	0.793	551
B	Riverview	0.576	1.05	728
C	9Th Street	1.37	2.83	1,968
D	Lagoon	1.01	2.06	1,430
E	Williams Street	0.446	0.861	598



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

FUTURE
SANITARY SEWER BASINS

HISTORIC INDEPENDENCE

FIGURE 4.2

4.3 Future Collection System Needs

Based on the projected basin flows, existing facilities, and potential future developmental patterns for the collection system were identified. Table 4.7 summarizes the projected peak flows and the capacities of the pump station and interceptors and was used to aid in determining the future collection system needs. The following sections describe the future collection system needs in greater detail. Detailed estimated budgetary costs and funding methods for these improvements are described and discussed in Sections 6 and 7. The location of the future collection system needs are shown in Figure 4.3 at the end of this section.

Table 4.7 Interceptor and Pump Station Capacity and Projected Basin Flows

Basin	Influent Pump Station	Interceptor Name	Manhole Location	Interceptor Capacity	Ultimate Buildout Peak Flow	Interceptor Qp/Qc	Existing Pump Station Flow	
				(mgd)	(mgd)		(mgd)	(gpm)
A	Oak St.	North	N6	1.0	0.793	0.63	1.06	740
			N11	1.39[3]	0.60		0.36[4]	250
B	Riverview	Confluence	M1	6.1	1.05	0.17	3.00	2080
		Middle	M2	0.72	0.354	0.49		
		South	S4	2.08	0.693	0.33		
C	9th St.	Confluence	M14A	4.80	2.83	0.59	1.30	900
		Monmouth	M14B	2.64	2.14	0.81		
		8th Street	M13-4	1.06	0.69	0.65	0.82[1] 1.00[2]	568 700
D	Lagoon	West	W5	1.32	2.05	1.55	2.17	1509
E	Williams	Airpark	A2	0.56	0.861	1.54	0.36	246

[1] Mt. Fir Pump Station 1 pump running
[2] Mt. Fir Pump Station 2 pumps running (room for 3 pumps)
[3] Sanitary Sewer Improvements: OR 51 Stryker Road to Polk Street
[4] New Hanna Road (North Main) Pump Station

4.3.1 Gravity Sewer and Interceptor Improvements

Analysis of current sewer system capacities and future requirements requires consideration of existing development, potential redevelopment, and undeveloped properties. Although redevelopment is likely for many parcels, it is also likely that redevelopment and new development will be of a similar nature and characteristics as the current land uses in terms of flows and loadings.

4.3.1.1 Basin A/North Interceptor

At buildout, the peak flows entering this interceptor from the industrial properties north of Polk Street (manhole N11), are projected to be 0.79 mgd. This exceeds the calculated hydraulic capacity of much of the old 8-inch interceptor. In the summer of 2006, as part of the Hwy OR 51: Stryker Road to Polk Street project, the City replaced the sanitary sewer from MHN11 to MHN24, and the North Main Pump station. The replacement sewer was sized to accommodate projected flows at ultimate buildout. The remaining 8-inch diameter segments of the North Interceptor, from MH N8 to N11, will need to be replaced as part of a future project but the need will be driven by future development. The capacity of this segment is estimated to be 1.0 mgd but must be verified with a topographic survey.

4.3.1.2 Basin B/South Interceptor

Under the current land use plan Basin B does not include large parcels of land that are undeveloped. Although there may be some infill and/or modifications to existing development, significant increases in base flow due to development are not anticipated. Flow generated within the basin is conveyed to the Riverview Pump Station. The basin is characterized by older homes and collection system components with notable amounts of I/I. The previously identified hydraulic bottlenecks identified in earlier studies have been eliminated as part of the 1998 Sewer Improvement Project. No critical capital improvements have been identified as part of this master plan. However, anecdotal reports from Public Works staff indicate that during extreme precipitation events portions of the interceptor flows full. A detailed hydraulic model and field measurements were beyond the scope of this report. City staff should monitor the condition and capacity of this interceptor and if it is determined that the pipe is flowing at capacity or surcharged, additional analysis and future capital improvements to replace the pipe may be warranted.

4.3.1.3 Basin C/Middle Interceptor

A large amount of land within Basin C is available for future development. Much of this is residential property located in the south end of the basin which will impact several sections of the downstream mainline. Improvements were made to the Middle Interceptor as part of the 1998 Sewer Improvement Project, and as part of the Mt. Fir Residential development. If development occurs as currently identified, additional improvements and replacement of the mainlines south of Monmouth Avenue will be required. The requirement for this work will be driven by private development.

4.3.1.4 Basin D/West Interceptor

Approximately 256 acres of land is available for new development within Basin D. Records of pipe sizes and slopes are available for the lower 1600 feet of pipe in the vicinity of the lagoons. However, upstream from MH W7 record drawings are not available to confirm the as-built geometry. If constructed in compliance with industry standards, there appears to be sufficient capacity to accommodate current and some modest future development. However, this pipe is relatively old and the structural condition could not be confirmed. The projected flows at basin build-out will exceed the current hydraulic capacity and will require replacement and this segment should be a high priority for replacement as part of the City's Capital Improvement Plan.

4.3.1.5 Basin E/Airport Interceptor

A large amount of land within Basin E is available for future development. The downstream segments of the interceptor are hydraulically inadequate to accommodate projected future development and will require replacement. The current peak flows are estimated to be 0.42 mgd with the limiting hydraulic segment (MH A-2 to MH A-3) having a full flow capacity of 0.56 mgd. At ultimate build-out the projected peak flow of 0.86 mgd will exceed the hydraulic capacity of the mainline. An estimated 3,510 feet of mainline, MH A-1 to MH A-8, will require replacement with larger and diameter pipe.

4.3.2 Pump Station and Lift Station Improvements

As a result of a focused capital improvement effort since 1998, most of pump stations serving large basins within the collection system have been improved to accommodate the projected ultimate peak flows. The exceptions are the Williams Street and 9th Street pump stations.

Williams Street is able to serve the current flows that are relatively small. However, projected future development at build-out will exceed the current capacity. An additional concern is that the station equipment is nearing the end of a reasonable design and service life and replacement of at least major equipment, will be required regardless of future development. Mechanical and electrical equipment were replaced in the fall of 2013 due to pump failure. It should be noted that this represents the second time that the station has been rehabilitated yet the original steel structure remains in service and is very close to reaching the limits of expected service life.

The 9th Street pump station was constructed as part of the City's 1998 Sewer Improvement project. The pump station's wet well and appurtenances were sized to accommodate the peak flows at ultimate build-out for the basin. However, due to the wide range between current flow demands and ultimate build-out, the pumps installed were sized to accommodate projected flows through 2018. This basin (Basin C) has the potential to

experience dramatic flow increases and growth, and associated hydraulic impacts to the pump station should be monitored. When peak flows begin to approach the firm pumping capacity (a single pump operating) at the 9th Street Pump Station, the pumps and motor controls should be upgraded. The existing pumps for Oak Street and 9th Street are the same motor size and impeller configuration. When the 9th Street Pump Station pumps are replaced, the existing pumps and motor controls can then be rebuilt and stored for use as back-up pumps for the Oak Street pump station.

The Williams Street Pump Station was construction in 1979 and is nearing the end of its useful life. As development occurs within Basin E, the firm pumping capacity will not meet the peak flows. Prior to the firm pumping capacity being met, the City should replace this pump station. The replacement will include lowering the pump station and constructing a wet well, valve vault, appurtenances, and force main to the WWTP headworks.

Other pump station improvements include the Maple Street and Briar Street pump stations which could be replaced with a gravity sewer and new pump station when development at the southern portion of the Mt. Fir subdivision is completed.

Within the UGB there are two areas where it appears additional pump stations and forcemains will be necessary to accommodate topographic limitations. This includes the recently annexed land in the southwest and northwest portions of the UGB. The need for these improvements will be driven by private development. Figure 4.3 provides a summary of the capital improvements that have been identified during the preparation of this report. This should not be considered a comprehensive master plan document as detailed collection system modeling and flow measurements was beyond the scope of this Facilities Plan. However, it does provide an initial outline of key areas that appear to be impacted. It is recommended that the City update the Wastewater System Master Plan to include a focused investigation and analysis of the collection system including updating the Capital Improvement Program.

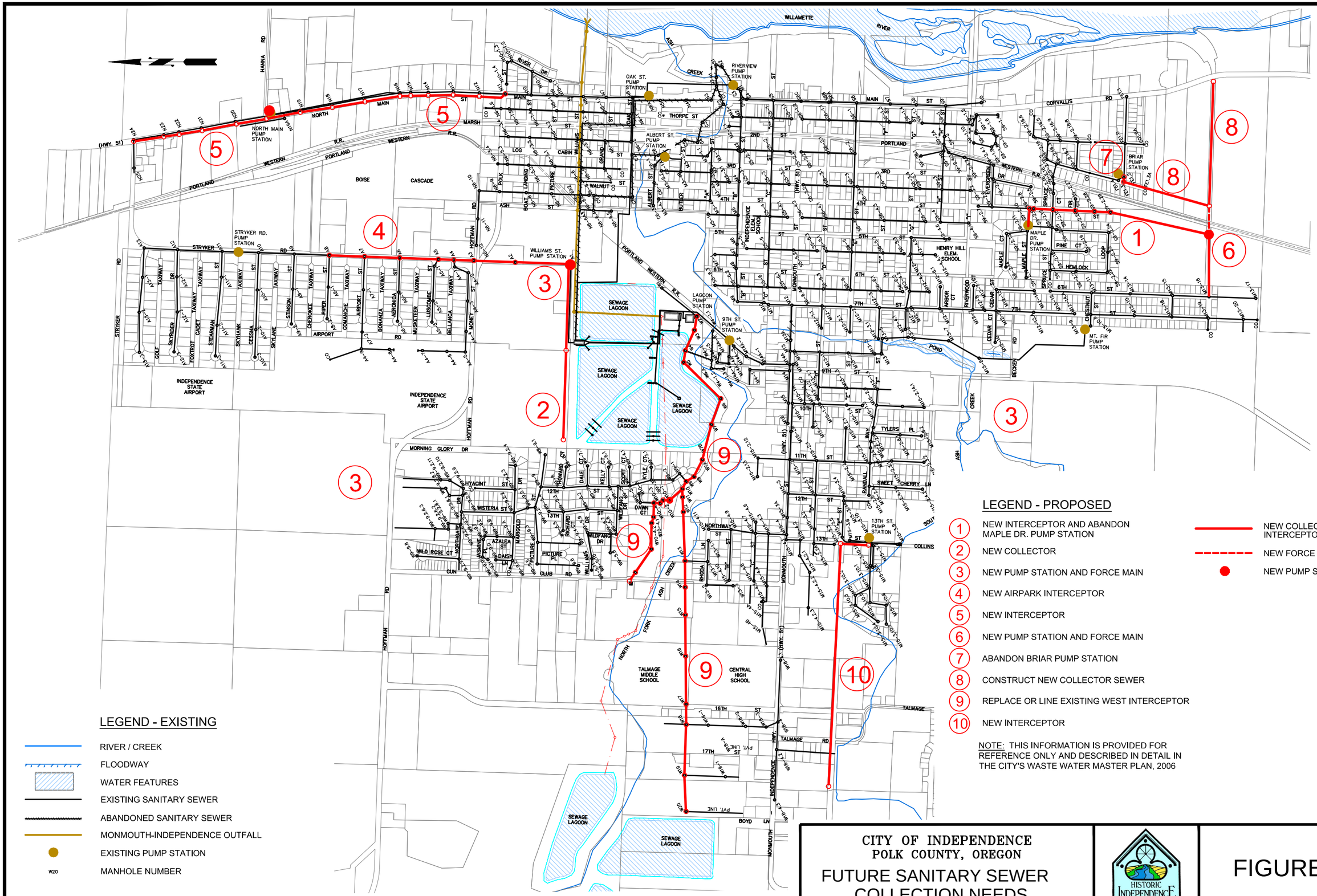
4.3.3 Collection System Infiltration and Inflow Improvements

As described in detail in Section 3.4 the general approach required for a successful I/I reduction program consists of (1) Establishing a baseline of Flow Measurements, (2) Addressing inflow and RII first, (3) Conducting a comprehensive program, (4) Establishment of a repair budget, (5) Conducting construction inspection and testing, and (5) Establishing an iterative process that monitors success and failures and adjusts future projects accordingly.

The City has taken several proactive steps toward implementing this approach since the city wide improvements project in 1998. In addition to a number of capital improvement projects, the City has implemented a proactive inspection and testing program to monitor new construction within the City jurisdiction. In addition, the City has conducted smoke

testing and television inspection, and has budgeted for the replacement of aging sewers. It is recommended that this program continue and includes all items described in Section 3.4.

The City should also anticipate and budget for the replacement of the private sewer laterals as part of mainline is replacement projects. One of the largest contributors of I/I flows is from service laterals. Comprehensive and targeted basin or sub-basin replacement including service laterals provides the most efficient and cost effective use of funds for reducing I/I.



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

FUTURE SANITARY SEWER COLLECTION NEEDS

HISTORIC INDEPENDENCE

FIGURE 4.3

5 WASTEWATER TREATMENT SYSTEM ALTERNATIVES

When projecting future flows and demands on the wastewater system two major components will require attention:

Biological Treatment

The existing facultative lagoons rely on transfer of oxygen between the air water surface that occurs passively. As such the ability of the lagoons to oxidize and reduce the biological waste is limited by the surface area of the lagoon cells. As a general rule of thumb an overall loading rate for primary lagoons of 35 lbs/acre/day for BOD is viewed as an upper limit beyond which consistent treatment cannot be assured. The current loading rate of the primary lagoons (Lagoon Nos. 1, 2 and 3) averages 36 lbs/acre/day. Accommodating future growth using lagoons will require additional treatment capacity either through additional surface area made available by expanding the lagoon area, or through mechanical aeration or oxidation. Although some residual treatment capacity may remain during average conditions, the City should begin the process of planning and constructing additional treatment facilities.

Summer Storage

Due to the permit requirements outlined in previous sections, the City is required to discontinue discharge to the Willamette River during the months of June through October (153 days). Current practice requires storing influent in the lagoons during this time period. During the no-discharge months some of the wastewater is lost to evaporation and allowed infiltration through the bottom of the lagoons. However, the vast majority accumulates and receives treatment until November when discharge to the Willamette River can once again commence. When the lagoons were new, under optimum conditions 76 million gallons (MG) of storage was available. Over time solids accumulation has reduced the volume available for storage by raising the minimum water elevation that can be achieved prior to the summer holding period. Based on DMR's for 2011 through 2014 the lagoons were able to store approximately 52 MG before reaching levels that were deemed to put the lagoons at risk. In every year since 2011 the City has requested and been granted authorization from DEQ to begin discharging treated effluent before the October 31 required holding period. DEQ has noted that as part of the new permit the option of early discharges will be removed.

In the Fall of 2009 a Lagoon Profile was completed by the Oregon Association of Water Utilities to measure the level of the solids accumulated in the bottom of the lagoons. In August 2012 a field survey was completed at the lagoons to measure and confirm the elevations of key features and the top of the sludge layer at

accessible locations. Applying these two surveys and dimensions from the original design drawings, sludge accumulation has reduced the total available storage volume by 24 MG (32%).

If the original storage volume could be recovered by dredging the lagoons, the additional 24 MG of storage would allow for a maximum of 2,090 additional persons. Based on a population growth rate of 2.8% this would be reached in the year 2023. This should be considered the most optimistic scenario and does not take into account the potential of large commercial or industrial users. Removing accumulated solids has additional value in that it frequently improves the biological performance of the lagoon system and potentially reducing odors.

To accommodate projected growth, expansion of the treatment facilities must address both of these limiting conditions (biological treatment and summer storage volume). To address the requirement for additional biological treatment five treatment alternatives will be considered. All five treatment technologies are well established with proven track records of success. Management of dry weather flows considered both increasing the volume of the existing lagoons, and developing alternatives for reuse of treated effluent through spray irrigation.

During the preparation of this Facilities Plan, DEQ representatives were consulted to assist with identifying potential and likely future permit requirements. DEQ traditionally does not provide formal review and comments until the draft report is submitted. As such, DEQ staff were not able to offer specific permit changes or anticipated additional requirements. However, they did offer some comments on current permit requirements in the context of current permit interpretations for similar community sewerage systems.

5.1 Objectives

Wastewater treatment alternatives must be based on the system's needs, but it is not sufficient to just select the least expensive alternative that will meet the permit requirements. A number of other City objectives must be also met including:

Accommodate growth – Provide capacity for growth over the next 20 years from the time the new system or improvements come online.

Meet DEQ requirements – For lagoon based treatment this currently includes; meet 85% BOD removal and 65% TSS removal requirements. Treat flows up to those resulting from a 5-year storm. Store or otherwise manage effluent 5 months out of the year. Have the ability to upgrade to remove nitrogen and/or other pollutants if required in the future.

Build affordable improvements – Construct the most cost-effective system which does the job. This will likely utilize the existing wastewater facility (lagoons) to the greatest extent possible.

Apply Appropriate Technology - Provide automated, operator-friendly, simple features that balance the use of new and innovative technologies without becoming an additional burden to the City staff.

Enhance open space and wildlife habitat – Where possible and for little or no additional cost, provide open space, wildlife habitat and public access.

Conserve water – Where economically feasible recycle or reuse treated wastewater effluent.

Conserve energy – Design facilities which minimize energy use for treatment processes including nitrification, de-nitrification, disinfection and pumping.

Serve as a Good Neighbor – Provide facilities which are compatible with adjacent properties and uses, including the residential area west of Lagoon Nos. 3 and 4, and public use areas and open space along Ash Creek.

Improve Public and Operator Safety – Protect facilities against unauthorized entry, water contact, and other illegal or unsafe activities by members of the public. Enhance operator safety.

5.2 Common Features of Alternatives

5.2.1 Meets Permit Requirements

All alternatives meet the requirements of the NPDES permit and OAR 340-55 Recycled Water Use.

5.2.2 Utilize Capacity of Existing Lagoons

Future storage requirements through the 20-year study period cannot be met soley by the existing facility. To meet future requirements will require implementing a strategy to either restore the original volume, or decrease the need for additional volume by expanding effluent reuse during the dry season. To take advantage of existing capacity, all alternatives must provide continued lagoon treatment/storage of 2015 sewage flows and loads. Under one approach the existing lagoons would be dredged and solids removed to restore the volume that has been lost to the accumulation of non-degradable solids. The City would continue wet weather seasonal discharge to the River when climate conditions render spray irrigation infeasible.

5.2.3 Additional BOD Treatment Capacity

All alternatives must provide additional wintertime BOD capacity to augment the facultative lagoon treatment capacity. At a minimum, approximately 946 lbs/day additional BOD

capacity must be added to accommodate growth from 2015 to 2030. The current loading rate to the lagoons is at the theoretic limit and it would be prudent to begin adding supplemental treatment as soon as the City is able to arrange for financing and complete design and construction. In 2035 1,355 lbs/day of additional BOD treatment beyond the current treatment capacity will be necessary if it is assumed the lagoons function at optimum level.

All alternatives must improve treatment capacity so that pollutant loads to the river remain below the current maximum limits. This should be accomplished by (1) effluent reuse and (2) a higher level of treatment for all additional flows and loads resulting from future growth.

5.2.4 Treatment Features Common to All Alternatives

All wastewater plant alternatives have these physical facilities:

Upgrade Existing Lagoons - Provide the existing lagoons with better effluent, decant effluent, and structural improvements.

Effluent reuse to agricultural or public lands – All alternatives have a recycled water clear well and an irrigation pump station.

Plant computer and SCADA system – All alternatives are controlled by a programmable controller with plant computers at the Shops and a Control Building on the site.

Control Building – A process, electrical and control building is provided in each alternative to monitor and control the lagoons and reuse treatment plant, and to perform basic lab analyses.

Headworks – A headworks with an influent screen and grit removal unit is required to remove screenings and grit from all raw sewage entering the plant. By reducing influent BOD and solids, building a headworks is one of the most cost-effective ways to maximize the life of the lagoons and the treatment facilities. As well, the headworks is needed to properly protect downstream mechanical treatment facilities, such as return sludge pumps. The headworks should be designed for the 2035 5-year peak average daily flow ($PADF_5$).

Influent Flow Control Structure – Downstream of the headworks, an influent flow control structure is required to split flow between the new treatment plant and the lagoons. If, for example, a flow of 1.5 mgd comes into the plant, the 2020 treatment plant expansion will receive its design flow of 0.50 mgd and the balance of flow, 1.0 mgd, will flow to the lagoons. The flow control structure should be designed to divert an additional 0.25 mgd to the Phase III facility to be constructed in 2030.

Disinfection Building Addition – An addition to the existing chlorine disinfection facility will be needed in each alternative to provide additional and replacement chlorinators, injectors and contact basin volume. Chlorine is preferable to UV as a

disinfectant because (1) the lagoon algae inhibits UV, and (2) For monitoring and compliance purposes, maintaining a residual chlorine level at the reuse points of application is desirable.

Irrigation Pump Station and Force Main – Each alternative has a recycled effluent clear well which serves the recycled water pumps. The pumps discharge through a recycled water force main piping to the reuse sites.

5.2.5 Maintains Permitted Pollutant Loads to River

All alternatives must produce effluent that remains within the permitted loads to the river including but not limited to total flow, thermal, mass loads, total solids, and biochemical oxygen demand.

5.2.6 Effluent Irrigation Compensates for Future Storage Requirements

Some additional summer storage volume may be achieved by removing accumulated solids and restoring the original design depth of the lagoons. However, the original storage volume was never intended to provide capacity for the population projected through the year 2035 and consequently a different strategy will be required to supplement the current lagoons. Reuse of treated effluent through irrigation is a cost effective long term strategy that will allow the City to accommodate future dry weather flows, and potentially reduce the demand on the potable water supply. It is recommended that the City focus capital improvements on developing a new recycled water (spray irrigation) facility with the flexibility to continue expanding beyond 2035.

5.2.7 Energy and Environmental Factors

All alternatives must meet U.S. energy and environmental goals to minimize energy and chemical consumption, operator hours, vehicle use, and construction in habitat, flood plain, or lands of historic/archaeological value.

5.3 Alternatives Considered Not Feasible

5.3.1 No Project Alternative

A no-project alternative is not considered feasible because sewage loads will exceed plant capacity unless improvements are made. The purpose of this facilities plan is to recommend a project which will solve this problem. If the City were to take no action, effluent violations would result and DEQ would take enforcement action.

5.3.2 Hyporheic Discharge Next to River

Hyporheic discharge is the discharge of treated effluent to a gravel stratum adjacent to and along a river, where near surface groundwater mixes with river water. This is not considered feasible because:

- City potable water wells are in close proximity;
- Extensive documentation and modeling is typically required;
- Since a high level of treatment is required for hyporheic discharge, discharging through the existing, permitted outfall would be equally beneficial and much less complex;
- Since a high level of treatment is required, effluent reuse would be feasible, with more benefits and less complexity than hyporheic discharge during dry weather months;

5.3.3 Expanding Existing Lagoon System

Building more lagoon area and storage capacity is an option for cities with lagoons far outside the urban growth boundary. For Independence, constructing a new lagoon is not considered feasible for the following reasons:

- The proximity of City residential areas on three sides of the facility (west, south and east).
- Expense and complexity of obtaining the needed 40 acres to the north of the existing facility.
- Loss of land which could be developed as residential, commercial or industrial purposes and would generate tax revenues.
- Zoning and state land requirements for the only available area, adjacent to the north boundary of the existing plant would require modifications to the current comprehensive plan and is inconsistent with current LCDC planning.
- Constructing a new lagoon would not significantly improve effluent quality, and wet weather flows through the river outfall would continue to increase. If DEQ does not allow increased mass loads to the river, further treatment in addition to the lagoon expansion will be required.
- If reuse is to be provided as part of this alternative, additional treatment will be required to meet Class A effluent requirements. If treatment facilities need to be constructed, effluent reuse would have many more benefits than expanding the footprint of the lagoons.
- The current facility does not provide optimal buffers with adjacent land uses and additional lagoons would decrease visual aesthetics.

- FAA and other aviation interests may oppose expansion of lagoon due to the attraction of water fowl and other birds.

5.4 Wastewater Treatment Process Alternatives

This section compares five wastewater treatment methods – aerated lagoons, conventional activated sludge, oxidation ditch, sequencing batch reactor, and membrane bioreactor. Treatment options are evaluated for their ability to provide:

- Recycled water during summertime to minimize storage requirements.
- Recycled water year-round for industrial, process, and other non-irrigation uses.
- Wintertime BOD treatment capacity to supplement the capacity of the existing lagoons.
- Treatment of potential future permit restrictions and emerging pollutants of concern (EPOC).

All five alternatives are assumed to address future growth and the addition of summer time effluent reuse. From this perspective it has been assumed that the existing facultative lagoons will remain an integral part of the City's wastewater management strategy during the next 20 years. An additional level of treatment will be provided to address growth and to provide a higher level of effluent treatment and disinfection which will offer a wider range of opportunities for spray irrigation.

A second assumption that is inherent to all five alternatives is the need for additional disinfection. The City currently uses chlorine gas which has been an effective and inexpensive approach. The chlorine injection system is nearing the end of its design life and replacement is assumed. In the coming years it is likely that the issue of chlorine toxicity at the discharge to the Willamette River will become a focus of permit discussions. With these two factors in mind it has been assumed that disinfection using chlorination/dechlorination facilities will be required under all treatment alternatives considered. As noted previously due to lagoon effluent characteristics, at this time disinfection using ultraviolet (UV) facilities is not considered to be feasible.

A final assumption common to all treatment approaches is the need to phase construction of new and additional treatment technology. The primary driver for phasing is the cost associated with the capital improvements.

Phase I – Immediate Improvements

The initial phase will address the immediate need for management of summer storage and will include the development of effluent reuse facilities to allow for recycling of treated and disinfected wastewater through spray irrigation on agricultural properties near the City's UGB.

Phase II – Additional Treatment Capacity

The second phase will address the future need for additional biological treatment to accommodate population growth through the year 2030. Capital improvements associated with this second phase are recommended to begin construction during the year 2020.

Phase III – Additional Treatment Capacity

The third phase will focus on the development of additional treatment and effluent reuse capacity to accommodate growth anticipated between the years 2030 and 2035. Construction of capital improvements associated with this third phase is anticipated to be required to begin during the year 2029.

Phased construction is recommended for these reasons:

- Building 3 to 15 years of capacity in phases, instead of 20 years in one phase makes the facility easier to finance.
- At this time, it is uncertain how much Independence will grow in 20 years. The City may double in size, or it may grow less than that. Building treatment capacity in two phases will more correctly size the facility to meet the real 2035 demands
- Wastewater reuse technology is improving every year. Twenty years ago, membrane filtration of wastewater was not an option, and now there are membrane facilities throughout the world. Waiting 15 years before building the Phase III capacity will allow the City to take advantage of equipment efficiencies and reduced capital costs.
- It allows for thoughtful evaluation of the most beneficial and cost effective effluent reuse. This may be through expanded development of summer spray irrigation facilities of agricultural land in close proximity to Independence, or development of reuse facilities within the City limits as part of future development of private and public facilities in close proximity to the WWTF.

Selecting an appropriate technology for the expanded and replacement wastewater treatment requires a comparative analysis of available processes. Five common and appropriate technologies will be evaluated and are briefly summarized in the following subsections.

5.4.1 Alternative No. 1 – Aerated Lagoons

Upgrading lagoon systems by aerating the primary cell(s) with floating mechanical aerators has been popular since the mid-1980s. With the addition of air, BOD oxidation capacity is increased. The process uses fixed air diffusers or floating mechanical aerators to inject air into the lagoon water, and mix the water column to provide uniform aeration, less light for algae photosynthesis, and oxidation of the soluble BOD.

The aerated area becomes a complete mix reactor with a high solids level which must be settled out in the downstream cells. By mechanically aerating the first and second cells, the remaining lagoons need to be supplemented or reconfigured to provide settling.

Some systems are adaptable to upgrade with aerators because they have the land area for a supplemental polishing lagoon. Other systems with online Class C or D reuse facilities do not require a high quality effluent with low solids. Aeration of lagoons can be accomplished using a wide variety of proprietary processes that fall into general categories as either floating or diffused aerators. A general schematic diagram of a typical aeration configuration applied to the existing lagoons is provided in Figure 5.1.

To use this treatment mode, the existing facultative lagoon system would be:

- Dredged to increase the overall volume and reduce the potential of re-suspending inert solids (at a minimum cell Nos. 1 & 2);
- Retrofitted to add aeration.

The advantages of an aerated lagoon treatment system are:

- Design –Utilizes existing wastewater treatment foot print.
- Effluent Quality –At least 20/20 effluent is possible with a good operator attention and steady flow. To get a reliable 10/10 or lower effluent, tertiary filtration will likely be needed.
- In some instances the initial capital cost of retrofitting an existing lagoon system is lower than that of constructing a new treatment facility.

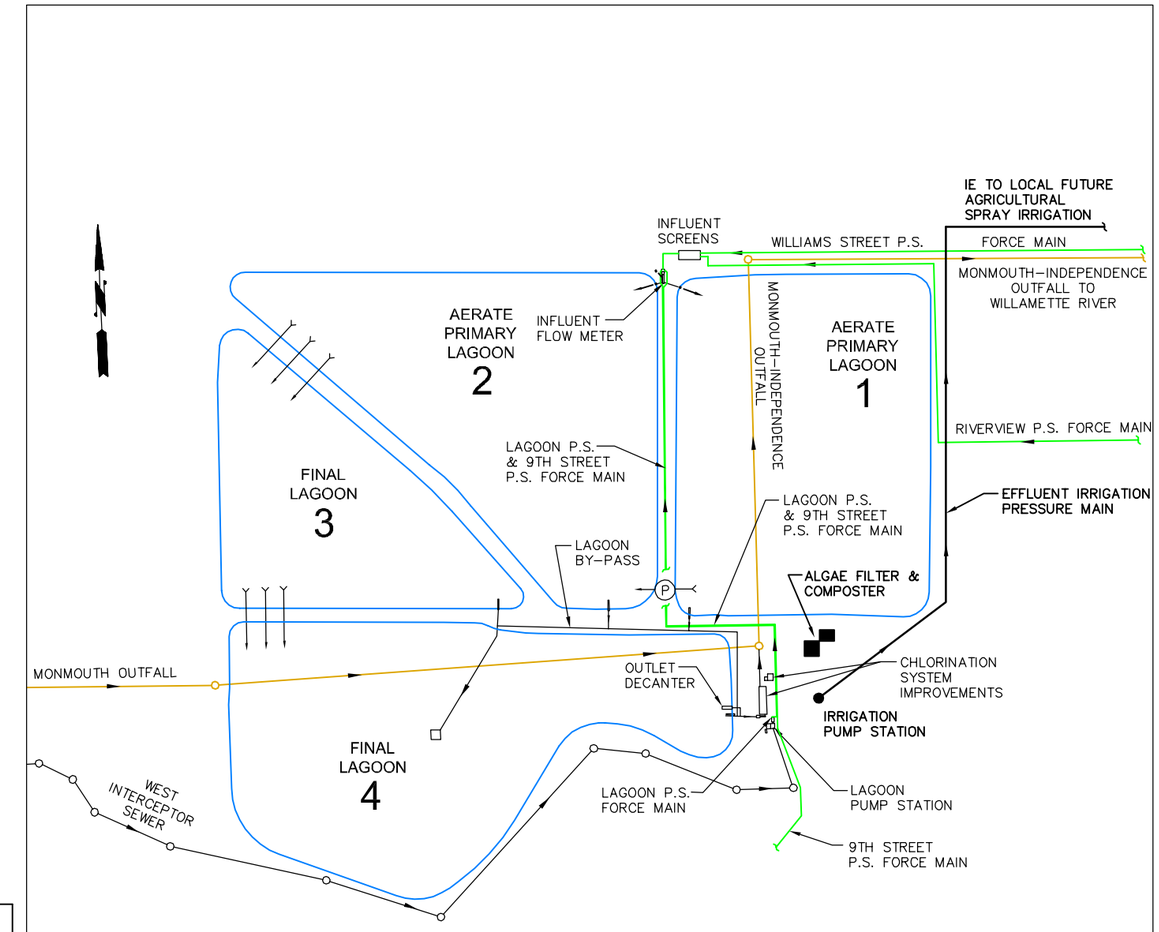
Disadvantages of an aerated lagoon systems are:

- Aerated lagoons are not as effective as facultative lagoons at removing ammonia nitrogen or phosphorus unless designed for nitrification.
- Aerating wastewater adjacent to residential areas can create odor problems that cannot be easily resolved.
- Some aeration processes create aerosols that are unsuitable with residential properties immediately adjacent.

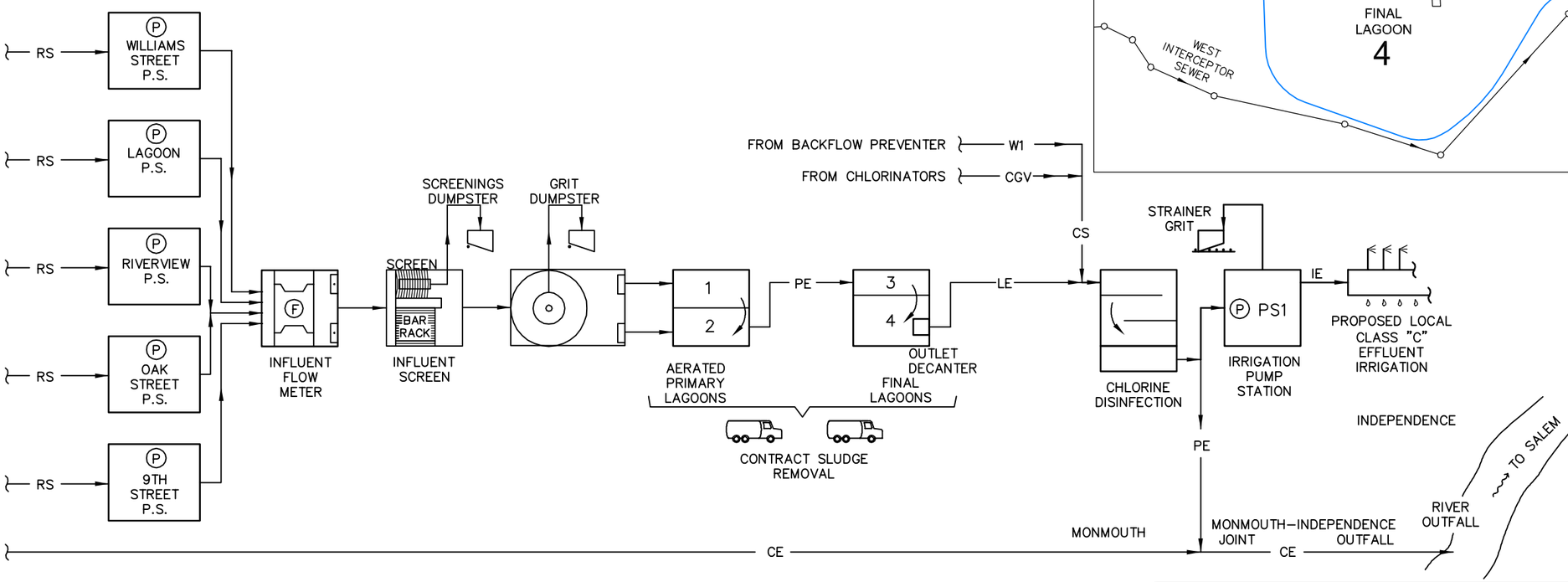
- Aerated lagoons without clarifiers do not have return sludge, waste sludge, mixed liquor, and other means of fine tuning process control that mechanical plants enjoy.
- Aerated lagoons defer solids handling for years before the sludge is removed by contract, incurring very large costs for pumping, dewatering, hauling and disposal, and impacting the treatment process with strong loads of centrate and supernatant. It could be argued that it is better to have a solids handling system, and pay as you go, than to leave a problem for future generations.
- For systems making Class C or Class D reuse water, as does the City of Monmouth, aerated lagoons are a feasible, low-cost way to oxidize the waste, chlorinate to meet the irrigation bacteria standards, and extend the life of existing lagoons. For Independence, aerated lagoons could not be easily upgraded to make significantly cleaner water for river discharge, or Class A reuse. For the City to expand capacity of an aerated lagoon system in 2030, the only option would be to provide more aerators and a way of reducing effluent solids. This lack of flexibility, compared to other treatment options, limits the City's reuse options.

LEGEND

CE	CHLORINATED EFFLUENT	PS1	IRRIGATION PUMP STATION
CGV	CHLORINE GAS VACUUM	PE	PLANT EFFLUENT
CS	CHLORINE SOLUTION	RAS	RETURN SLUDGE
DS	DIGESTED SLUDGE	RS	RAW SEWAGE
F	FLOW METER	SDS	SULFUR DIOXIDE SOLUTION
FM	FORCE MAIN	WAS	WASTE SLUDGE
IE	IRRIGATION EFFLUENT	W1	WATER – POTABLE
(P)	PUMP		



SITE PLAN



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CITY OF INDEPENDENCE
POLK COUNTY, OREGON

TREATMENT ALTERNATIVE 1
AERATED LAGOONS

FIGURE 5.1

5.4.2 Alternative No. 2 – Activated Sludge Treatment

Figure 5.2 is a process schematic of a typical activated sludge plant. In conventional activated sludge (CAS) systems, the incoming sewage is mixed with activated sludge in an aeration basin. The mixed liquor, at a density of about 4,000 mg/l flows to the clarifier, where sludge, scum and effluent are separated by gravity. The activated sludge which settles at the bottom of the clarifier is returned to be mixed with more incoming sewage, with a portion "wasted" to the sludge system.

Oxidation ditches, SBR's, and aerated lagoons followed by clarifiers are all activated sludge systems. Membrane bioreactors are also a type of activated sludge system, differing from the others because solids separation in an MBR is through a membrane barrier. CAS, oxidation ditches, SBR and aerated lagoon/clarifier systems all rely on gravity settling, or clarification, to separate the mixed liquor solids from the effluent.

Activated sludge systems can achieve good clarification and as high as "5/5" (BOD/TSS) effluent, provided they are run by skilled operators who understand how to maintain the sludge. To get this effluent quality, operators must be proactive in maintaining the sludge. Operators must always attend to the sludge settleability in the clarifiers as conditions change, by analyzing the solids balance and the sludge volume index, adjusting the return and waste sludge rates, changing the depth of the clarifier blanket, characterizing the microorganisms in the sludge, and other process tasks.

For Independence, to provide maximum utility for reuse and river discharge, a CAS plant would be designed to achieve a 10/10 effluent. The advantages of a CAS treatment system are:

- Design - Engineered design customized to the site conditions and design criteria of selected equipment manufacturers.
- Effluent Quality - 10/10 effluent possible with a proactive operation and steady 0.5 mgd flow. To get a reliable 5/5 effluent, tertiary filtration would be needed.
- Expandability in 2030 – CAS can be expanded by building an additional basin and clarifier, sludge pumping and aeration system. Digester and sludge storage could be doubled in size. Basin can easily be converted to biological nutrient removal (BNR), if required for river discharge.

Disadvantages of a CAS system are:

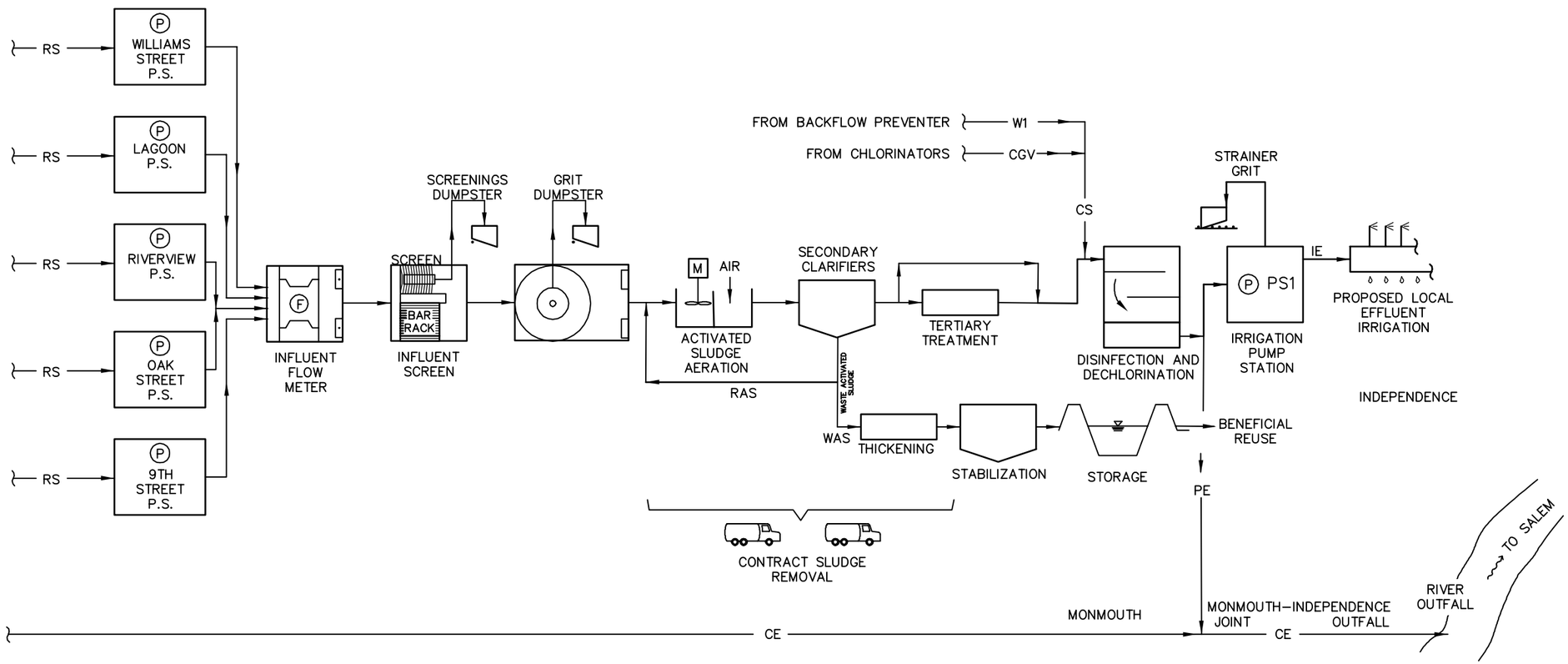
- Complexity - Requires operator experience in performing the various tasks to control the process, troubleshoot problems, maintain the sludge, produce high quality effluent, and operate the solids handling system.
- The engineer and owner, who determine the equipment to be used, inherit responsibility for how well it works. In contrast, manufacturer responsibility is

provided for proprietary systems such as SBRs, some types of oxidation ditches, and MBRs.

- Sludge Maintenance - Activated sludge requires process monitoring in addition to the DEQ monitoring, and a laboratory is desirable.
- CAS systems tend to be loud with relatively high energy demands.
- Capital Cost - Construction costs are similar to costs for an oxidation ditch, and higher than costs for SBR or MBR systems, which do not require clarifiers.
- O&M Costs – Requires (2) qualified full-time operators. Material costs are the same as for the SBR, oxidation ditch and MBR to control pH, alkalinity, residual chlorine and ammonia.

LEGEND

CE	CHLORINATED EFFLUENT
CGV	CHLORINE GAS VACUUM
CS	CHLORINE SOLUTION
F	FLOW METER
IE	IRRIGATION EFFLUENT
(P)	PUMP
PS1	IRRIGATION PUMP STATION
PE	PLANT EFFLUENT
RAS	RETURN SLUDGE
RS	RAW SEWAGE
WAS	WASTE SLUDGE
W1	WATER – POTABLE



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CITY OF INDEPENDENCE
POLK COUNTY, OREGON

TREATMENT ALTERNATIVE 2
ACTIVATED SLUDGE

HISTORIC INDEPENDENCE

FIGURE 5.2

5.4.3 Alternative No. 3 – Oxidation Ditch

Oxidation ditch systems (Figure 5.3) are identical to CAS systems, but use a somewhat simpler type of aeration basin, consisting of a large oval tank fitted with disc or brush aerators. Whereas a conventional aeration basin has fixed air diffusers to aerate and mix the mixed liquor, an oxidation ditch can perform these functions more efficiently, by using the motive power of the mechanical aerators to circulate the mixed liquor around the oval tank. As with activated sludge systems, a clarification process is needed downstream of the oxidation ditch to separate the effluent from the scum and settleable solids.

Oxidation ditch activated sludge systems can achieve good clarification and a “5/5” BOD/TSS effluent, provided they are managed by an operator who understands all the duties of an activated sludge plant, described in the previous section.

As for the CAS plant, the oxidation ditch plant would be designed to achieve a 10/10 effluent. The advantages would include those for the CAS treatment system, with some added attributes:

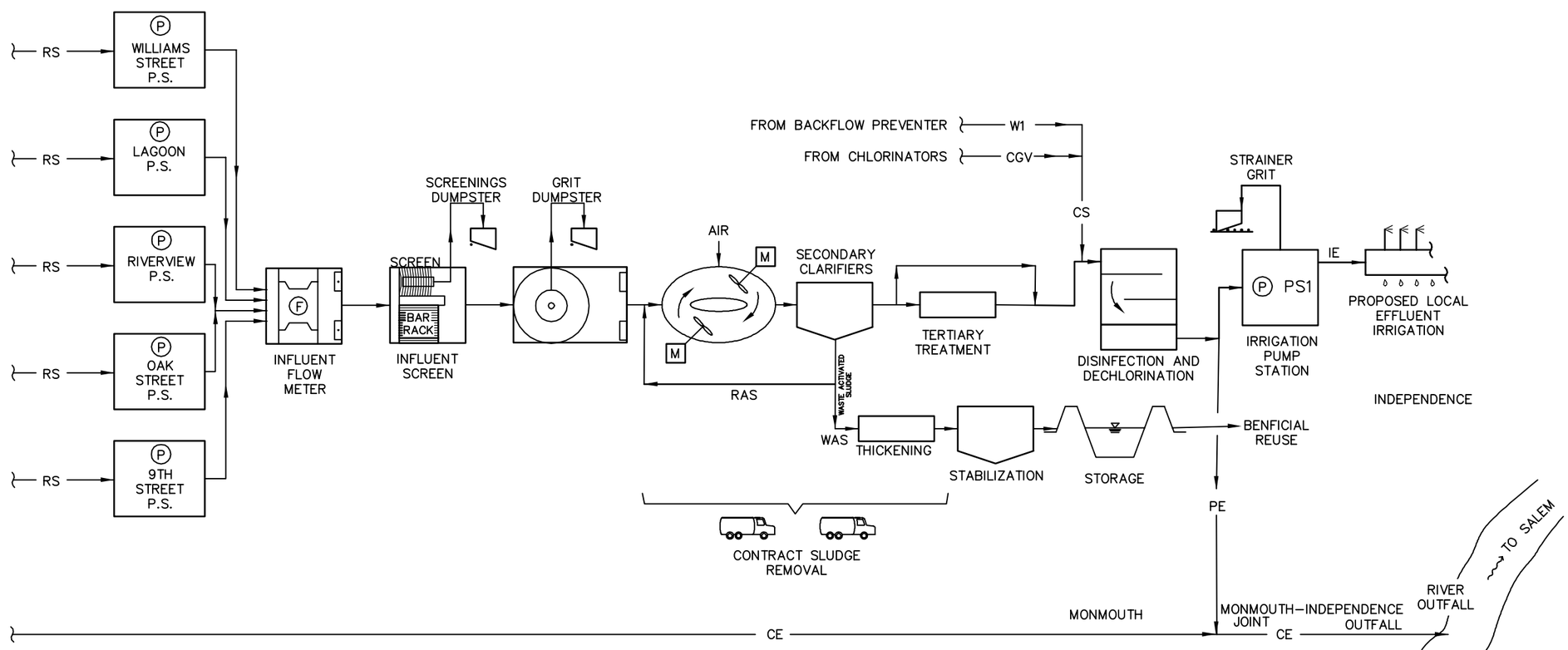
- Improved Reliability Against Shock Loads – Oxidation ditches have longer hydraulic detention times compared to most aeration basins, and are better able to absorb shock loads without a plant upset.
- Produces Less Sludge – Oxidation ditches create less sludge solids than do CAS aeration basins. Ditches have longer detention times, resulting in extended aeration and more complete breakdown of the solids
- Custom Design – Oxidation ditches are engineered designs customized to site conditions. Oxidation ditch equipment manufacturers generally recommend size and dimensions of the tank.
- Mixing, Aeration and Energy Efficiency – An oxidation ditch does not need submerged air diffusers or process air blowers, which consume most of a treatment plant’s energy. The brush aerators, provided with variable speed drives, require less power for mixing and aeration than do blowers and diffusers.
- Tank Efficiency – Oxidation ditch tanks are much easier to clean than aeration basins. Diffuser fouling with reconstituted rags and sludge deposition is not a problem.
- Effluent Quality - 10/10 effluent possible with a conservatively designed oxidation ditch and clarifier, experienced operators, and steady 0.5 mgd flow. To get a 5/5 effluent, tertiary filtration would be needed.
- Expandibility in 2030 - Can be expanded by building an additional oxidation ditch, clarifier, sludge pumping and aeration system. Digester and sludge storage could be doubled in size. The 2030 oxidation ditch could be configured to provide biological nutrient removal if required for river discharge.

Disadvantages of an oxidation ditch activated sludge system are:

- Complexity – About the same as for the CAS system, although the oxidation ditch is more forgiving and easier to maintain than an aeration basin. Similar to CAS and SBR systems, it requires activated sludge operator experience in performing the various tasks to control the process, troubleshoot problems, maintain the sludge, produce high quality effluent, and operate the solids handling system.
- Sludge Maintenance – Oxidation ditch activated sludge requires process monitoring in addition to the DEQ monitoring, and a laboratory is desirable.
- Relatively high back-up power requirements due to large blower capacity.
- Capital Cost - Construction costs are similar to costs for a CAS plant, and higher than costs for SBR or MBR systems, which do not need clarifiers.
- O&M Costs – Requires (2) qualified full-time operators. Material costs are the same as for the CAS, SBR, and MBR systems to control pH, alkalinity, residual chlorine and ammonia.

LEGEND

CE	CHLORINATED EFFLUENT
CGV	CHLORINE GAS VACUUM
CS	CHLORINE SOLUTION
F	FLOW METER
IE	IRRIGATION EFFLUENT
(P)	PUMP
PS1	IRRIGATION PUMP STATION
PE	PLANT EFFLUENT
RAS	RETURN SLUDGE
RS	RAW SEWAGE
SDS	SULFUR DIOXIDE SOLUTION
WAS	WASTE SLUDGE
W1	WATER – POTABLE



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CITY OF INDEPENDENCE
POLK COUNTY, OREGON

TREATMENT ALTERNATIVE 3
OXIDATION DITCH

FIGURE 5.3

5.4.4 Alternative No. 4 – Sequencing Batch Reactor (SBR)

SBRs, as shown schematically in Figure 5.4, consist of two identically configured tanks. The tanks alternate cycles with one tank filling and aerating while the second tank settles and decants (clarifies) the effluent. SBRs use the activated sludge process of mixing raw sewage with organism-rich, aerated sludge, which is separated from the effluent by gravity settling in a quiescent, unmixed clarifier tank. Unlike CAS and oxidation ditch plants, SBRs have no clarifiers. Instead, for clarification, SBRs automatically stop the aeration, wait for settling and separation to occur, and activate effluent decanters which withdraw clarified effluent.

The advantages of an SBR treatment system are:

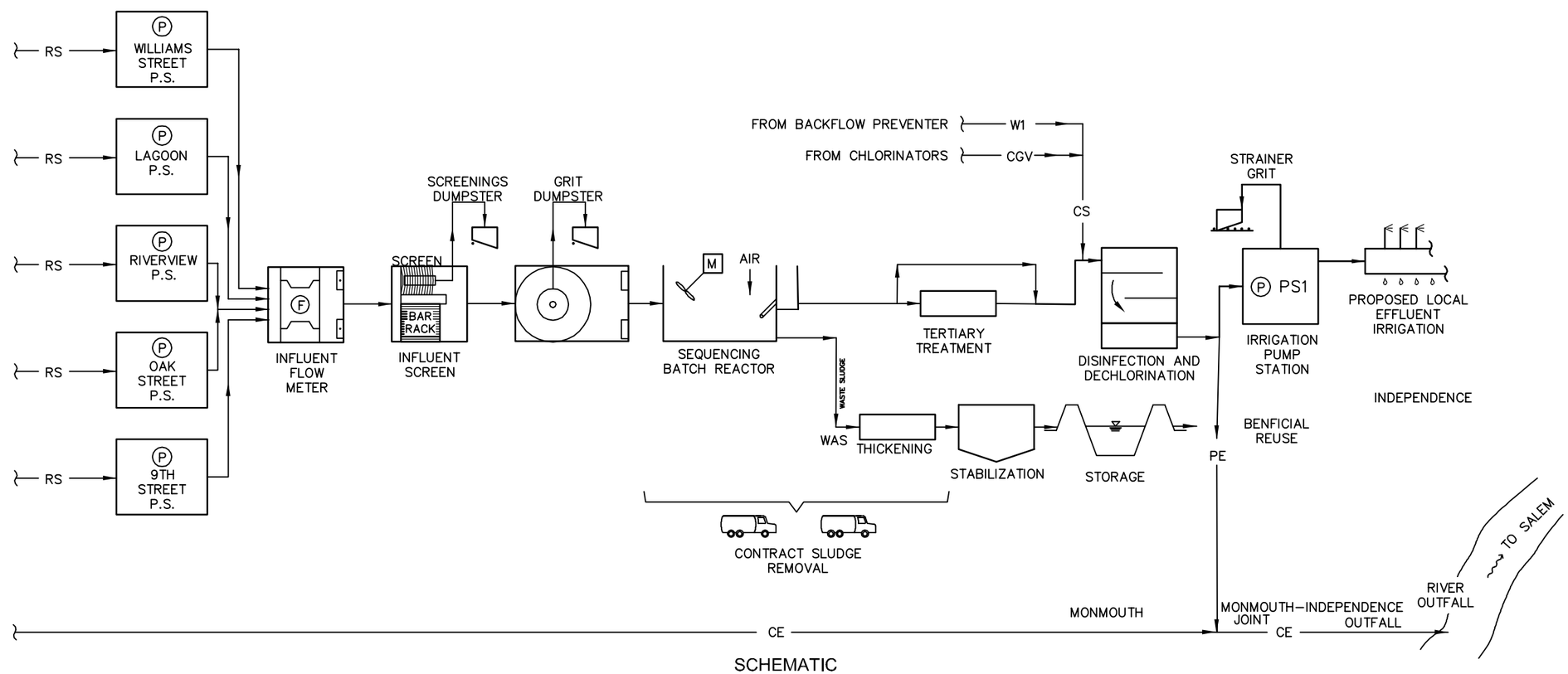
- Design – Like the other alternatives, SBRs are engineered designs customized to site conditions, with assistance from equipment suppliers. There are several principal SBR equipment manufacturers, who dictate the size and dimensions of the tank, provide the equipment, troubleshoot and warranty equipment.
- Automation – Compared to CAS and oxidation ditch treatment systems, SBR processes are somewhat more automated and compact. Automating treatment plants makes them easier to operate, and allows operators to concentrate on optimizing the process rather than perform duties that can be done electronically. An SBR's process control computer cycles on pumps, blowers and equipment needed to aerate, clarify and decant the effluent.
- No Clarifier Required – The SBR tank and automated effluent decanter is used for clarification. Very good clarification, equivalent to that of a conventional clarifier, can be achieved in an SBR, provided that the sludge settles well.
- Effluent Quality - 10/10 effluent is possible with a conservatively designed SBR with flow equalization (which the City's existing lagoons provide). One SBR operator in Oregon reports that their Austgen-Biojet SBR gets 5/5 effluent. However, since this plan assumes tertiary equipment is needed to get 5/5 from a CAS or oxidation ditch system, this is also assumed for the SBR. Since SBRs are proprietary, the need for tertiary treatment might be eliminated if the manufacturer is willing to guarantee a 5/5 effluent.
- Expandability in 2030 - Can be expanded by building additional rectangular SBR basin structure similar to the 2020 structure. Each structure includes the two SBR basins, sludge pumping, aeration system, aerobic digester and sludge storage cells. SBRs can be configured to provide nitrogen removal (denitrify) if needed for river discharge.

Disadvantages of SBRs, compared to the other treatment systems, are:

- Complexity – In addition to understanding activated sludge, operators must understand how to maintain the SBR automation and control system. Electrical and mechanical expertise is critical in troubleshooting. To provide high quality effluent, the activated sludge settleability must be maintained. Compared to the other systems, the SBR has the greatest complexity. Because the systems run well, however, the added complexity is often considered to be worth it.
- Sludge Maintenance – SBR aerobic sludge digestion and storage cells, sludge transfer pumps and a truck loading facility are needed, as for the other treatment processes. SBR activated sludge requires process monitoring in addition to the DEQ monitoring, and a laboratory is desirable.
- SBR's may have a problem with large variations in influent flow.
- Relatively high back-up power requirements due to large blower capacity.
- Capital Cost - Construction costs are less than costs for a CAS or oxidation ditch plant because a clarifier is not needed. SBRs and MBRs have similar equipment costs, but SBR equipment, which is largely mechanical and electrical, will probably last longer between replacements than will membranes. SBRs require slightly more tank volume than do MBRs.
- O&M Costs – SBR operation requires 1 to 2 qualified full-time operators. Material costs for controlling pH, alkalinity, residual chlorine and ammonia are the same as for the CAS, oxidation ditch and MBR systems.

LEGEND

CE	CHLORINATED EFFLUENT
CGV	CHLORINE GAS VACUUM
CS	CHLORINE SOLUTION
F	FLOW METER
IE	IRRIGATION EFFLUENT
(P)	PUMP
PS1	IRRIGATION PUMP STATION
PE	PLANT EFFLUENT
RAS	RETURN SLUDGE
RS	RAW SEWAGE
WAS	WASTE SLUDGE
W1	WATER – POTABLE



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CITY OF INDEPENDENCE
POLK COUNTY, OREGON

TREATMENT ALTERNATIVE 4
SEQ. BATCH REACTOR (SBR)

HISTORIC INDEPENDENCE

FIGURE 5.4

5.4.5 Alternative No. 5 – Membrane Bioreactor (MBR)

The MBR process, shown schematically in Figure 5.5, is also an activated sludge process. It differs from the processes which need a clarifier, and it differs from the SBR by relying on membrane separation of the effluent from the solids instead of settling.

There are several proprietary processes that are available and typically fall into one of two general categories: immersed “flat plate” membrane units, or reinforced hollow fiber membranes.

Advantages of the MBR treatment system compared to the other options are:

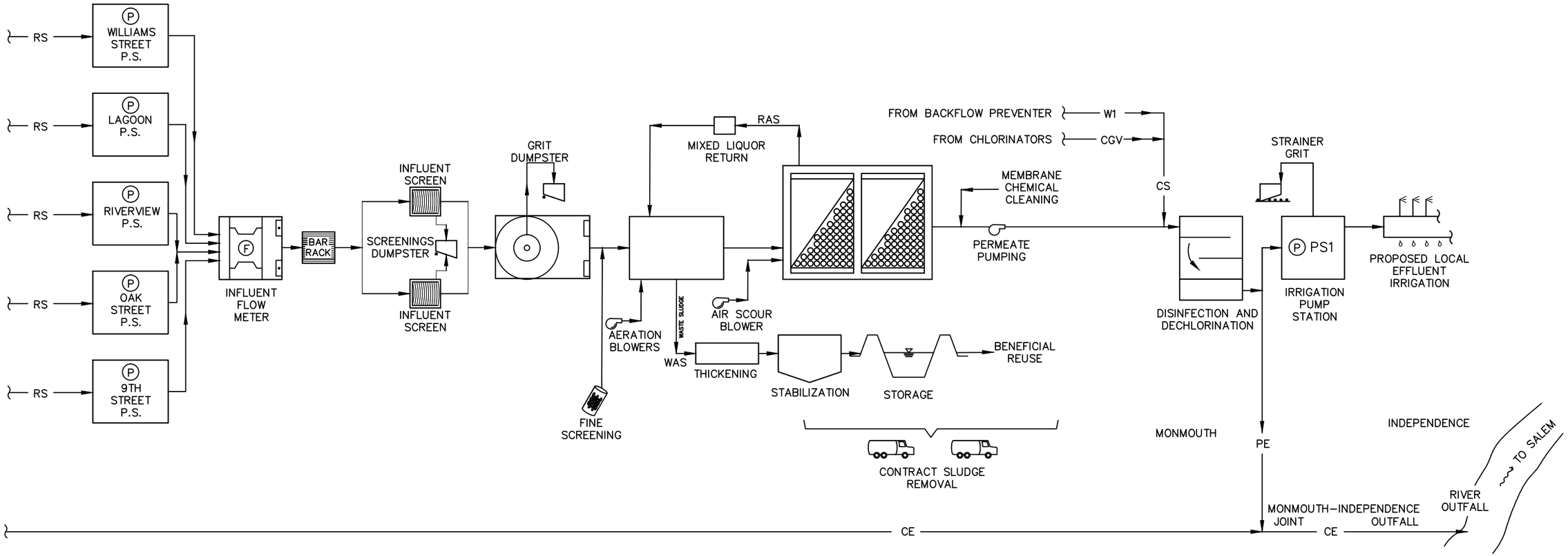
- Design – Like the other alternatives, membrane bioreactors are engineered designs customized to site conditions, with assistance from equipment suppliers.
- Effluent Quality - 5/5 or better effluent and Class A reuse water is reasonable to expect with a conservatively designed MBR with flow equalization. MBRs are proprietary and a manufacturer may guarantee Class A effluent provided that appropriate disinfection is provided.
- Relatively small footprint.
- Automation – MBRs are highly automated. MBR control and troubleshooting are comparable to an SBR but slightly simpler because there are fewer moving parts. The MBR process computer cycles on pumps, blowers and equipment needed to mix and aerate the mixed liquor, and the permeate pumps needed to pass the clarified effluent through the membrane.
- Less Complex than Other Systems – Maintenance of the activated sludge settleability is not important for MBR systems. Compared to SBR systems, MBR’s are less complex.
- No Clarifier Required – The MBR’s membrane filters clarify the effluent and no clarifier or settling tanks are required.
- Expandability in 2030 - Can be expanded by placing additional membrane units in the 2020-built MBR basin structure. An additional structure will be required for sludge digestion, storage, and sludge transfer pumping. An additional blower and RAS/WAS pumps would be added to the 2020-built process rooms. MBRs can be configured to denitrify if needed for river discharge.

Disadvantages of MBRs, compared to the other treatment systems, are:

- Complexity – MBR’s are less complex than the many other systems, but they still require considerable expertise. Operators must control mixed liquor suspended solids (maintained at much higher density in MBRs than for other activated sludge systems), waste sludge as needed, and control the solids handling process.

Operators must understand how to maintain the MBR automation and control system.

- Membrane Fouling – The most serious problem associated with MBRs is membrane fouling and decreased filtration capacity. Air scouring past a maximum flow rate is not effective at eliminating fouling. The best way to minimize fouling is to regularly clean the membranes per the manufacturer protocol.
- Need for Membrane Cleaning – Membranes must be periodically cleaned, although this is reportedly a fairly easy task. Flat plate membranes are reported by some operators to be easier to clean than hollow fiber membrane systems. Flat plate membranes are cleaned in place, using a bleach solution for cleaning.
- Membrane Replacement – MBR membranes must be periodically replaced.
- Blower Energy Use – To function properly, membrane units must be continuously blasted with process air from coarse bubble diffusers. Blower energy use, which is the principal power consumption item in wastewater plants, is higher for MBRs than for other activated sludge units, which use fine bubble diffusers or mechanical aerators.
- Relatively high back-up power requirements due to large blower capacity.
- Sludge Maintenance – Aerobic sludge digestion and storage cells, sludge transfer pumps and a truck loading facility are needed, as for the other treatment processes. The activated sludge process and DEQ monitoring make a small laboratory desirable.
- MBR's may have problems with large influent variations.
- Capital Cost - Construction costs are less than costs for the processes requiring a clarifier. SBRs and MBRs have similar equipment costs, but MBR equipment will probably last longer.
- O&M Costs – MBR operation requires 1 to 2 qualified full-time operators. Material costs for controlling pH, alkalinity, residual chlorine and ammonia are the same as for the CAS, oxidation ditch, and SBR systems.



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LEGEND

CE	CHLORINATED EFFLUENT	PS1	IRRIGATION PUMP STATION
CGV	CHLORINE GAS VACUUM	PE	PLANT EFFLUENT
CS	CHLORINE SOLUTION	RAS	RETURN SLUDGE
F	FLOW METER	RS	RAW SEWAGE
IE	IRRIGATION EFFLUENT	WAS	WASTE SLUDGE
(P)	PUMP	W1	WATER – POTABLE

CITY OF INDEPENDENCE
POLK COUNTY, OREGON

TREATMENT ALTERNATIVE 5
MEMBRANE BIOREACTOR (MBR)

HISTORIC INDEPENDENCE

FIGURE 5.5

5.4.6 SBR VS. MBR

Advantages of using an SBR (with gravity settling) instead of an MBR are:

- Proven Technology – Settling basin design has been tested by time. The SBR decanters perform well.
- Cost – For the size of plant envisioned during this design period, a SBR plant may be constructed at a cost somewhat less than an MBR plant.
- No Membrane Cleaning or Replacement - SBR's do not require membrane cleaning and replacement.
- SBR Performance Guarantee – SBRs are a proprietary product, and at this time, multiple manufacturers provide SBR systems and may be expected to guarantee and take responsibility for adequate performance.
- SBR plants require less energy than MBR's.

Advantages of an MBR plant over an SBR plant are:

- Also a Proven Technology - There are many successful MBR systems throughout the northwest and the world. Membranes, the newest big wastewater technology, are increasingly popular as water becomes scarce and more wastewater reuse programs come online. In the Independence area, the 0.3 mgd Spirit Mountain Casino MBR plant is a good example of the type of MBR system which could serve the City.
- Effluent Quality – The flat plate MBR filters are Title 22 approved, and can produce Class A water which can be used in a wide variety of reuse sites and applications. With some exceptions, MBR plants may be expected to produce a higher quality effluent than SBR plants.
- Less Sludge Production – Extended aeration of sludge is provided by the long solids retention time in the MBR tank.
- Membrane Provides Physical Barrier - MBR plants do not rely on activated sludge settleability, and don't require the detailed attention to sludge management and monitoring that is required in plants using clarifiers. The membranes work reliably at MLSS levels from 8,000 to 18,000 mg/l.
- In-Place Cleaning Not Difficult - MBR's require periodic cleaning, but flat plat membrane units are easily cleaned in-place without the need for a toxic or corrosive cleaning agent.
- Ease of Disinfection – The MBR effluent quality requires less chlorine, or, if UV disinfection is used, less UV dosage power to get the required bacterial kill.
- MBR Performance Guarantee – Membrane bioreactors are a proprietary product, and manufacturer's guarantees performance.

- Membranes Better Remove Trace Constituents –Testing results for side-by-side activated sludge and MBR pilot plants have demonstrated that MBR plants showed superior removal of trace constituents, including pharmaceuticals, endocrine-disrupting compounds and other micro-constituents.

5.4.7 Flat Plate vs. Hollow Fiber Membranes

Although both technologies are in use, discussions with plant operators during the preparation of this plan noted a preference for flat plate membranes over hollow fiber membranes for these reasons:

- Process Control – Flat plate membrane systems are simpler and easier to operate.
- Membrane Fouling and Cleanability – Hollow fiber membranes, cleaned by backpulsing are more difficult to clean than flat plate units.
- Footprint – Flat plate MBRs occupy somewhat less overall space than hollow fiber systems.
- Side and Recycle Streams – Flat plate systems are reported to be simpler in function without the need for sidestream screening, and requiring fewer recycle streams.
- Peak Flow Capacity – Flat plate membranes can better absorb peak flows and loads than can the hollow fiber systems.

Hollow fiber MBRs are appropriate when a very high quality effluent is needed. GE Water & Process Technology's ZeeWeed advertises an effluent BOD / TSS / Total-N of 2/.5 / 3; several of their installations are designed for 5/5/10. Flat plate membranes perform nearly this well, but their overall resistance to fouling is a significant value for municipal wastewater treatment. Between now and 2030, when additional membrane units are added, technology of both types of membrane is expected to keep advancing.

5.5 Disinfection Alternatives

Disinfection processes considered are:

Continue Use of Existing Chlorination System

1. Chlorination with gas; dechlorination with sulfur dioxide gas.
2. Chlorination with gas; dechlorination with sodium bisulfite
3. Chlorination with gas; dechlorination with calcium sulfite tablets.

Upgrade Existing Chlorination System

1. Chlorination with sodium hypochlorite; dechlorination with sodium bisulfite.
2. Ultraviolet disinfection

Other Dechlorination Options

Dechlorination is only needed for river discharge of chlorinated effluent. Dechlorination can also be achieved with other methods, but each has its drawback, which eliminates it from further consideration:

Dechlorination Method	Drawback
1. Ferrous Sulfate	Corrosive; produces a floculant that requires disposal
2. Hydrogen Peroxide	Cost; Works better on free chlorine than chloramines
3. Sunlight UV	Reduces chlorine too slowly, only works when sunny and a storage pond required

5.5.1 Chlorination/Dechlorination

5.5.1.1 Chlorination with Gas and Dechlorination with Sulfur Dioxide Gas

Chlorination with Gas – This disinfection option would continue use of the existing gas chlorination, with minor improvements to improve safety and monitoring. Whereas using chlorine gas is perceived as dangerous by some people, many operators are accustomed to using chlorine gas and feel comfortable with the equipment and safety procedures. For the Independence plant, the City operators are experienced in using the chlorine gas system and maintaining the chlorine safety features.

Dechlorination with Sulfur Dioxide - Sulfur dioxide gas is popularly used for dechlorination, particularly for larger plants. In a process analogous to the existing chlorination system, sulfur dioxide is metered through a “sulfonator” (nearly identical to a chlorinator) then pulled into a moving column of water in a sulfur dioxide injector, which uses the water pressure and velocity to create a vacuum. Drawing the SO_2 from the tank under vacuum is much safer than sending pressurized SO_2 into the effluent.

For the Independence plant, the inherent danger of sulfur dioxide and the small flow make this option more expensive and less safe than dechlorination with bisulfite or metabisulfite.

5.5.1.2 Chlorination with Gas and Dechlorination with Sodium Bisulfite

When using chlorine for disinfection, the objectives are to (1) chlorinate the effluent enough to get a good bacteria kill in the contact chamber, and then (2) remove nearly all of the residual chlorine.

Chlorination with Gas – This option would continue the use of the existing gas chlorination system.

Dechlorination with Sodium Bisulfite - Sodium bisulfite is the dechlorination system of choice for many small plants, and it is recommended for the Independence plant in the event that the DEQ permit residual chlorine limit is decreased when the permit is renewed. $NaHSO_3$ is a white powder which is available in solutions up to 44 percent. It takes 1.5 parts of bisulfite per 1 part of chlorine to complete the reaction:

$$NaHSO_3 + Cl_2 + H_20 \rightarrow NaHSO_4 + 2\ HCl$$

For each part chlorine removed, 1.38 parts of alkalinity as $CaCO_3$ is consumed, so it is important to check the effects on effluent pH. Typically, lowered pH is not significant as long as the water contains sufficient alkalinity.

Sodium Metabisulfite, a similar dechlorinating agent, can also used. Each part of chlorine residual consumed requires 1.34 parts of sodium metabisulfite, and 1.38 parts of alkalinity are consumed:

$$Na_2S_2O_5 + 2Cl_2 + 3H_20 \rightarrow 2NaHSO_4 + 4HCl$$

The byproducts of both dechlorinating agents are sodium, hydrogen and chloride ions, and sulfuric acid. The sulfuric acid dissociates in water to hydrogen and sulfate ions. All of these constituents, in the quantities which would occur in the effluent, are considered harmless to aquatic life and people.

A small submersible pump would draw carrier water from the chlorine monitoring manhole. Sodium bisulfite, pumped from a carboy or drum by a chemical feed pump, would be injected in the chlorinated effluent. The chemical feed pump of choice is a very controllable Grundfos metering pump with an 800:1 turndown ratio.

To dose the effluent at 0.75 mg/L sodium bisulfite would require about four pounds per day, or 600 lbs of 44 percent solution annually. It degrades during prolonged storage, particularly in warm temperatures, so it is best to buy small quantities. Unlike sulfur dioxide gas, a separate storage room isn't required for sodium bisulfite.

5.5.1.3 Chlorination with Sodium Hypochlorite and Dechlorination with Bisulfite

Chlorination with Sodium Hypochlorite - Chlorine gas is inherently hazardous because it is destructive to respiratory passages and heavier than air. Chlorine gas is commonly used for disinfection because chlorine equipment mitigates the hazards by using vacuum transport piping and adequate chlorine leak alarms, and a protocol for chlorine system design and inspection.

In many plants where chlorine gas safety was a concern it was replaced with sodium hypochlorite, an industrial grade of bleach solution about 4 times as strong as household bleach. A 12.5 percent solution contains about 1.0 pound of chlorine per gallon. The solution is delivered in 55-gallon drums or 350-gallon totes, and added to wastewater using chemical metering pumps. The rate of solution strength decay increases with temperature, exposure to light, or the presence of metal catalysts. The storage volume of hypochlorite should not exceed 28 days.

If the Independence plant used this disinfection option, a hypochlorite tank and chemical feed pump would replace the existing gas cylinder and chlorinator. The capital cost of changing over would be low; however, the disadvantages lay in the higher chemical cost, degradation with time; and maintenance responsibilities. Unless the City operators dislike gas chlorination systems, there is no reason to change.

5.5.2 Ultraviolet Light Disinfection

UV disinfection, in which effluent passes through a zone of intense, UV light, is popular with plants who are chlorine safety conscious and have a near-zero residual chlorine standard. UV disinfection of lagoon effluent is only practical for use by lagoon facilities with final polishing cells, filters and other features to remove lagoon algae and other suspended solids. DEQ notes that at this time there are no lagoon systems in Oregon using UV disinfection.

5.5.2.1 UV and Lagoon Effluent

UV disinfection works best on an effluent with high transmissivity, low color and low suspended solids. UV's effectiveness is limited by lagoon algae. The most commonly means of lagoon algae removal is a process of dissolved-air flotation followed by filtration through a self-cleaning upflow media filter such as the Hydrasand unit. Without these secondary / tertiary additions to the treatment chain, Independence and other lagoon plants would run a risk of bacteria violations during unfavorable periods when lagoon algae limits transmissivity and fouls bulbs.

UV's advantages are:

- Neither chlorination or dechlorination are required
- Chemical storage/ handling isn't needed
- Simplicity of operation.

The drawbacks of UV for the lagoon system are significant:

- UV may not work (achieve the bacteria kill) unless effluent quality and transmissivity are improved.
- A pilot study would be needed to assess if UV is effective.

- Capital cost for added effluent process and UV structure.
- Higher O&M costs for power, bulb cleaning and replacement.
- Need for 480V, 3-phase power service.

5.5.2.2 UV and Treatment Plant Effluent

For new treatment plants with good effluent quality, UV disinfection is a popular choice. For Independence, UV disinfection isn't a good choice because:

- UV disinfection could only be provided for the new mechanical plant, while lagoon effluent continues to be chlorinated.
- Reuse water requires residual chlorine to retain its low bacteria levels at the point of application. UV can't maintain a residual to prevent bacteria re-growth in the effluent pipe.
- The City is relying on reuse as an integral part of its wastewater plan. Chlorination is more irrigation-friendly than UV because a residual chlorine level can be maintained in the distribution system to keep bacteria levels below the OAR 340-55 limits.

5.5.3 412 Chlorine Gas Safety Requirements

Chlorine is a highly toxic gas. The Uniform Fire Code (UFC) allows the use of toxic or highly toxic gases where treatment systems (scrubbers) or containment systems are provided. The UFC has an exception to this requirement for scrubbers or containment, when gas detectors and automatic shut off valves are provided. *UFC Section Treatment Systems 63.3.8.3.2* allows the storage and use of 150 pound cylinders and ton containers <u>without</u> the requirement of scrubbers or other containment systems when an automatic, fail-safe shut off valve is employed in the facility at the tank valve. The storage facility must have:

- A gas detection system with a sensing interval not exceeding 5 minutes.
- An approved automatic-closing, fail-safe located immediately adjacent to active container, cylinder or tank valves.

If the UFC didn't have this exception, expensive chlorine gas scrubbers would be required wherever chlorine is used, and chlorine gas would not be an economical choice.

Chlorine Gas Tradeoffs - The advantages of chlorine gas are:

- Low equipment capital cost (excluding scrubber)
- Lower chemical cost than hypochlorite
- Simple O&M
- Stability in storage

Chlorine gas disadvantages are:

- Requirement for dechlorination

- Potential safety hazard to operators and public in the event of a release
- Need to develop a Risk Management Plan for EPA and conform to UFC

Sodium Hypochlorite Tradeoffs – The advantages of sodium hypochlorite are:

- Lower capital cost than for chlorine gas
- Simple O&M
- Stringent EPA and UFC requirements don't apply

The disadvantages of sodium hypochlorite are:

- Dechlorination is required
- Higher chemical cost than chlorine gas
- Stored sodium hypochlorite loses strength over time, producing chlorine fumes
- Liquid chlorination feed systems require more maintenance than gas chlorination systems

In the Independence plant, the chlorine gas system is existing and paid for – a "sunk" cost. Since most of this system can be used for many years to come, both the capital and annual costs of gas is less than hypochlorite.

Added Cost vs. Improved Safety - The additional cost for hypochlorite can be considered the cost for improving safety against chlorine leaks. Because the operator does not believe using chlorine gas is risky, the additional cost is not worth it, and continued use of gas chlorine is proposed.

5.6 Water Reuse Alternative

Disposal of treated effluent is particularly critical during the summer months when relatively low flows in the Willamette River prohibit discharge. The City currently stores flows during the months of June through October. This approach will become increasingly infeasible as projected population growth will require extremely large areas of land to be dedicated to additional lagoon cells. A more practical approach with additional side benefits is the development of effluent reuse including summer spray irrigation.

Treated and disinfected effluent can be applied as recycled in a wide variety of applications. The range and type of areas where this is allowed is dependent on the level of treatment and disinfection provided. Increasing the level of treatment through one of the five processes previously described, will allow for application on a wide range of land uses including crops, open spaces, public parks and landscaping.

The City of Monmouth has successfully developed recycled effluent for agricultural land south of the Monmouth city limits. A similar recycled effluent option is available to

Independence by developing separate facilities at a location closer to the current and proposed WWTF.

For agricultural and industrial applications with low risk of human contact, Class C and Class D water may be either made through a treatment plant, or using lagoon effluent. For irrigating non-contact areas, Class D effluent requires somewhat less treatment and monitoring than does Class C. However, producing Class C water is preferred for these reasons:

- Class C allows many more in-City uses, including effluent irrigation of processed food crops, orchards and vineyards, golf courses, cemeteries, highway medians, industrial and business campuses.
- Class C can be made either from raw sewage through the treatment plant; or from the lagoon effluent. Lagoon effluent can be used for Class C irrigation if the algae is removed, generally by dissolved air flotation (DAF) and filtration.
- If facilities are designed only for Class D effluent reuse, the City's future options for reuse with Class C or Class A water are limited until another plant upgrade is completed.

5.7 Spray Irrigation Alternatives

Applying OAR 340-055 *Recycled Water Use*, Oregon Department of Environmental Quality (DEQ) regulates spray irrigation of treated wastewater. Although details are provided in OAR 340-055, three important factors that should be considered at a planning level are: type of treatment and disinfection, allowable uses, and setback or buffer requirements. The following table provides a general summary of these parameters:

Table 5.1 Summary of Spray Irrigation Parameters

Spray Irrigation Treatment Requirements					
	Class A	**Class B**	**Class C**	**Class D**	**Non-Disinfected**
Biological Treatment	X	X	X	X	X
Disinfection	X	X	X	X	
Clarification	X	X			
Coagulation	X	X			
Filtration	X				
Allowable Uses					
	Class A	**Class B**	**Class C**	**Class D**	**Non-Disinfected**
Agricultural	All	Most	Most	Few	Few
Urban/Irrigation	Yes	Some	Some	No	No
Commercial/Industrial	Yes	Yes	Yes	No	No
Construction	Yes	Yes	Yes	No	No
Impoundment	Yes	Some	Some	No	No
Public Access Restrictions and Buffer Requirements					
	Class A	**Class B**	**Class C**	**Class D**	**Non-Disinfected**
Public Access	None	No Contact During Irrigation	No Contact During Irrigation	Controlled	Prevented
Buffers		10’	10’/70’	Surface 10'/100’, Spray Site specific	Site specific

The selection of appropriate spray irrigation sites is dependent on a number of factors including:

- Soil type
- Crop type
- Level of Treatment and disinfection
- Potential ground water impacts
- Proximity to surface water and potable water sources.
- Intended land use
- Characteristics of the wastewater (phosphorus, nitrogen)
- Time of year for irrigation
- Growing season
- Climate

The soil type and crop or vegetation are two of the most significant in determining the application rates which translate directly to the amount of area required. Agricultural crops are particularly sensitive to application rates with specific windows of opportunity for

irrigation and tightly controlled application rates. Irrigation of landscaped areas, golf courses, parks and athletic fields often provide a more predictable and consistent demand since they are seasonal and relatively constant.

Regardless of the nature of the crop or landscape material, a storage volume is required to allow for buffering the continuous generation of treated effluent, with the cyclic demand of irrigation. Generally agricultural crops require a larger volume to match their less frequent irrigation cycles. The storage volume can be made available through construction of tanks, or earth impoundments. Independence is in a fortuitous position and land is available adjacent to the current wastewater treatment facility that could be redeveloped as impoundment. Another alternative is the reconfiguration of the existing lagoons to provide a cell dedicated solely to the storage of treated and disinfected wastewater. A final and least expensive option is to treat and use recycled water on demand drawing directly from the chlorine contact chamber.

Final sizing of the storage volume and hydraulic components of a spray irrigation system will require careful consideration and analysis. Whenever possible, design contingencies should be included to allow the City to adapt to changing demands of potential irrigation sites. It is in the City's long term interest to keep the system as flexible as possible, with a wide variety of potential irrigation sites and geographic areas. This allows the system to adapt to a variety of land development, climate, seasonal and economic scenarios.

The amount of area required for application is the direct result of application rates. In developing the alternatives for treatment and disposal typical application rates were used for the soils, land uses and crops generally in use within the study area. For purposes of developing approximate land area requirements the United States Soil and Conservation Study for Polk County, Oregon was consulted to identify suitable crops and approximate irrigation application rates. Additionally, the *Monmouth/Independence Joint Effluent Reclamation Project Feasibility Study, September 2004 (Whitaker Engineering, Inc.)* was reviewed.

The primary purpose of developing a spray irrigation program to dispose of treated and disinfected effluent, is to provide for the disposal of effluent during the dry weather holding period. The current NPDES permit requires that the City discontinue discharges to the Willamette River from the period of June 1st through October 31st (153 calendar days). During this period all influent is stored in the facultative lagoons. As noted previously as a result of population growth and expanded commercial and industrial interests, the City is at the limit of the available storage volume. At a minimum, recycled effluent facilities will need to be developed to accommodate flows associated with future growth. In practice, development of recycled effluent facilities should allow for an anticipated transition to recycling the majority of summer flows, current and projected by the year 2035.

5.7.1 Land Requirements for Spray Irrigation

As noted previously irrigation can be applied through a variety of uses. The following is a general summary of potential irrigation development and the appropriate net areas that will be required to allow for development of irrigation for the proposed development.

Irrigation for agricultural crops is dependent on the type of crop and growing season. Because the application rate is critical to the successful cultivation of a marketable crop it is important that an adequate volume of storage is available to buffer the continuous production of treated effluent from the periodic irrigation demand. The following is a summary of typical application rates and land requirements associated with crops in the Willamette Valley. These values should be considered general in nature and final selection of a site and sizing of associated irrigation components should be selected based on a refined irrigation and hydraulic calculations, a Recycled Water Use Plan, and a preliminary design report.

Table 5.2 Land Requirements for Typical Crops

			Land Requirements			
	Net Irrigation in/yr	Net Irrigation gal/acre/yr	2025 Excess Volume Stored mg	2025 Area Required acres	2035 Excess Volume Stored mg	2035 Area Required acres
Alfalfa	16.3	442,584	75	169	118	266
Filberts	22.0	597,353	75	125	118	197
Grapes	10.1	274,239	75	273	118	430
Grass Seed	18.6	505,035	75	148	118	234
Pasture	18.4	499,604	75	150	118	236
Pears	19.8	537,618	75	140	118	219
Poplar Trees	42.0	1,140,401	75	66	118	103

The values note for “Excess Volume Stored” in Table 5.2 represent total volumes including both existing and projected ADWF assuming an average year precipitation. All effluent discharge options require continued seasonal wet weather river discharge, which is necessary to operate the lagoons.

5.7.2 Class C Agricultural Reuse and River Discharge

Class C reuse to agricultural lands requires a low degree of treatment, which can be provided by the existing lagoons or with some supplemental aeration. The City of Monmouth uses this type of system. The reuse pipelines are long, and would only go to large tracts of land outside the City.

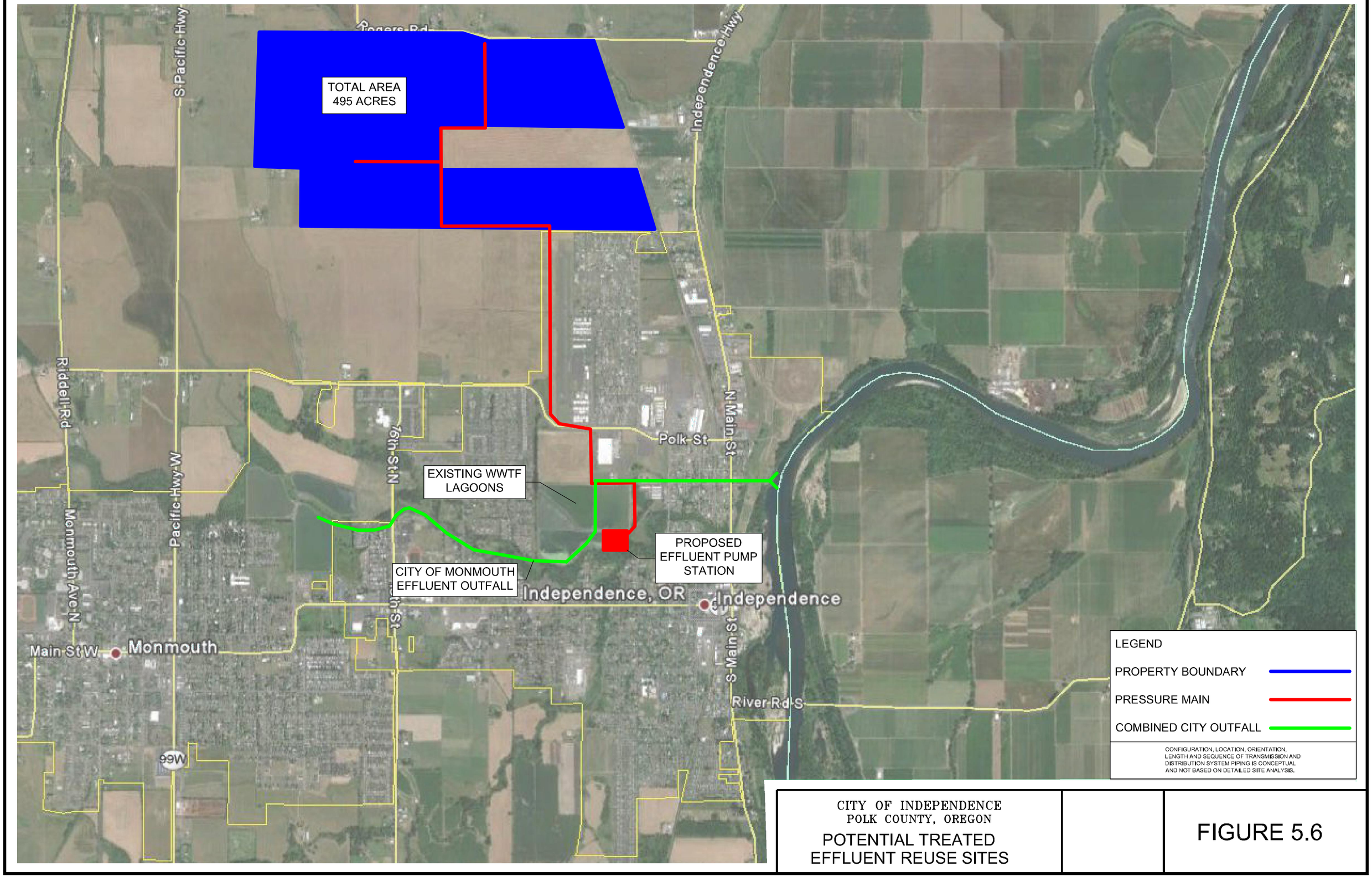
In the summer of 2013 City staff contacted several owners of property in close proximity to the city Urban Growth Boundary to determine their interest in receiving recycled water (treated and disinfected effluent) for irrigation. Lafayette Farms, Inc. expressed an interest to irrigate approximately 495 acres (gross area) of land. During subsequent meetings and communications the City and Lafayette Farms have signed a Letter of Interest outlining the intent of both parties to move to a formal agreement. The location and approximate areas of the parcels are shown in Figure 5.6. This figure also shows a potential alignment of the pressure main and pump station that would be used to supply recycled water to these areas.

The advantages of this type of system are:

- Low treatment cost.
- Effluent nutrients delivered to agricultural fields and potentially reduce the need for supplemental fertilizer application.

Disadvantages are:

- Long transport pipelines
- Treatment system would not provide Class C or A effluent suitable for urban reuse.



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

POTENTIAL TREATED
EFFLUENT REUSE SITES

FIGURE 5.6

5.7.3 Class C Recycled and River Discharge

Class C reuse to golf courses, industrial parks and campuses can be provided from lagoon effluent. In some instances high suspended solids including algae requires using a dissolved air flotation (DAF) clarifier followed by a backwashed filter. By providing this equipment in the treatment plant, effluent for Class C sites can be generated for most lagoon based treatment facilities without the need to pass it through the mechanical treatment plant.

The advantages of this type of discharge option are:

- Treatment cost lower than for activated sludge treatment alternatives..
- Transport pipelines to in-City areas are shorter than those to agricultural areas.

Disadvantages are:

- Treatment system doesn't provide Class A effluent for in-City reuse in public areas such as parks.
- Providing the plant with a DAF clarifier and filter for Class C reuse of lagoon effluent will be economical only if sufficient Class C sites are available.
- DAF clarification/filtration does not provide added BOD capacity

5.7.4 Urban Reuse and River Discharge

Class A effluent can be used in the above locations as well as any agricultural or horticultural use; landscaping of parks, playground, school yards, residential and other publicly accessible landscaping. Class A effluent can be used for nonrestricted recreational impoundments, landscape fountains, and to wash cars.

Advantages of treating and reusing Class A effluent are:

- Class A effluent can be used in a multitude of urban areas, as described above, with less restrictive boundaries and buffer areas required to protect the public.
- Shorter transport pipelines to in-City areas. Class A reuse pipelines could follow some existing City sewage force main routes, where easements and right-of-way have been established.
- A new treatment plant is needed in order to develop additional BOD capacity. Since a modern plant will be constructed, it makes sense to design it for a minimum 5/5 effluent, with the flexibility to improve technology. A plant designed for 5/5 can be expected to provide Class A reuse water.
- In the wintertime discharge period, the existing lagoons will operate at capacity, sometimes discharging solids levels above 30 mg/l. A plant which makes 5/5 Class A effluent can be used for effluent blending with the lagoon

effluent. For example, if the lagoons discharge at 0.7 mgd of 30 mg/l TSS, splitting half of the flow to a new treatment process capable of producing 5 mg/l plant effluent would produce a final blended effluent concentration of 17.5 mg/l.

Disadvantages of Class A treatment and reuse are:

- Higher treatment cost than for producing Class C or D water from the lagoons.

5.7.5 River Discharge

This alternative would construct a treatment plant which meets BOD/TSS limits of 10 mg/l monthly average and 15 mg/l weekly average, residual chlorine levels below 0.05 mg/l, and continue seasonal discharge to the river. The City would request that DEQ allow discharge of high quality flows during June through September. October discharge will not be sought to protect oxygen sensitive fish.

Based on preliminary discussions with DEQ obtaining a permit modification allowing for a dry weather discharge is unlikely. The burden of proof would be on the City to justify that the impact of discharging treated effluent to the Willamette River is beneficial, or at least prove that it does not degrade the water quality. This process could be lengthy with no guarantee of success. For these reasons seeking a summer discharge is not a recommended solution.

6 COST ESTIMATES AND ALTERNATIVE COMPARISION

This section compares the alternatives and improvements identified in previous sections, and recommends a strategy by which the best aspects of alternatives can be implemented. To ensure that the City only builds what is needed at the time it is needed, construction in phases over time is an essential part of these recommendations.

6.1 Comparison of Wastewater Treatment Alternatives

6.1.1 Goals

Minimizing cost and saving money is an important factor when comparing alternatives, but it is only one of the goals. To compare and evaluate the five alternatives, it is useful to apply the project goals and objectives developed in Section 5. All of the alternatives are feasible, and can be implemented in a manner that will achieve the plan goals.

6.1.2 Cost Comparison

Cost estimates developed in this chapter are planning level cost estimates based on manufacturers' cost quotations, standard engineering cost estimating practices, and similar projects.

6.1.3 Decision Matrix

A Decision Matrix of Alternatives was developed and shown in Table 6.1. Decision matrices are summaries that are often used to compare alternatives. The advantages are sometimes converted to a numerical system to facilitate decisions.

Sanitary Sewerage System Facilities Plans are intended to make it easier for decision-makers and individuals in the City to choose between complex alternatives. In developing this facilities plan the consultant team has developed a "decision matrix," comparison tables, and illustrations, but their overall purpose is to assist, and not mandate to, the decision-makers and the public. The matrix can be a useful tool in the comparison and development of consensus for the most appropriate system and future facility improvements.

Based on the criteria and weighting provided in the table, Alternative Nos. 4 and 5 (SRB and MBR treatment) receive significantly higher rankings than other alternatives. This is primarily due to the fact that these two treatment technologies offer significantly better treatment efficiencies, greater flexibility to meet changing regulatory requirements, utilize a smaller footprint, and allow for a greater range of recycled effluent options.

Table 6.1 Decision Matrix - Wastewater Treatment Alternatives

	Evaluation Criteria	Weighting of Importance	BENEFIT IN IMPLEMENTING GOAL				
			Alternate No. 1	Alternate No. 2	Alternate No. 3	Alternate No. 4	Alternate No. 5
			Lagoon Aeration	Activated Sludge	Oxidation Ditch	Sequencing Batch Reactor (SBR)	Membrane Treatment (MR)
1	Ease of Accommodating Phased Growth						
	Raw Score		1	1	1	2	2
	Weighted Score	100%	1	1	1	2	2
2	Ease of Permitting						
	Raw Score		1	0	0	0	0
	Weighted Score	100%	1	0	0	0	0
3	Effectiveness at Achieving Effluent Water Quality						
	Raw Score		1	1	1	2	2
	Weighted Score	100%	1	1	1	2	2
4	Reliability						
	Raw Score		1	1	1	1	1
	Weighted Score	100%	1	1	1	1	1
5	Initial Capital Cost to Build (Affordable)						
	Raw Score		1	-2	-2	-1	-1
	Weighted Score	300%	3	-6	-6	-3	-3
6	Improve Site Aesthetics						
	Raw Score		-2	-1	-1	2	2
	Weighted Score	100%	-2	-1	-1	2	2
7	Environmental Impacts						
	Raw Score		1	1	1	2	2
	Weighted Score	100%	1	1	1	2	2
8	Conserve Potable Water						
	Raw Score		1	1	1	2	2
	Weighted Score	100%	1	1	1	2	2
9	Conserve/Generate Energy						
	Raw Score		-1	-1	-1	-2	-2
	Weighted Score	100%	-1	-1	-1	-2	-2
10	Annual Cost to Operate (Rates)						
	Raw Score		-1	-2	-1	-2	-2
	Weighted Score	100%	-1	-2	-1	-2	-2
11	Integrate with Open Space						
	Raw Score		-2	0	0	2	2
	Weighted Score	100%	-2	0	0	2	2

	Evaluation Criteria	Weighting of Importance	BENEFIT IN IMPLEMENTING GOAL				
			Alternate No. 1	Alternate No. 2	Alternate No. 3	Alternate No. 4	Alternate No. 5
			Lagoon Aeration	Activated Sludge	Oxidation Ditch	Sequencing Batch Reactor (SBR)	Membrane Treatment (MR)
12	Administrative Efficiency for Combined Facility						
	Raw Score		0	1	1	1	1
	Weighted Score	100%	0	1	1	1	1
13	Solve Immediate Capacity Problems						
	Raw Score		1	0	0	1	1
	Weighted Score	100%	1	0	0	1	1
14	Land Requirements/ Opportunities						
	Raw Score		0	0	0	2	2
	Weighted Score	100%	0	0	0	2	2
15	Accessibility to Additional Funding Sources						
	Raw Score		0	0	0	1	1
	Weighted Score	100%	0	0	0	1	1
16	Adaptability to Future Regulatory Changes						
	Raw Score		-2	1	1	2	2
	Weighted Score	100%	-2	1	1	2	2
17	Community Support						
	Raw Score		1	0	0	1	1
	Weighted Score	100%	1	0	0	1	1
18	Utilization of Existing Staff/Certifications						
	Raw Score		0	-1	-1	-1	-1
	Weighted Score	100%	0	-1	-1	-1	-1
19	Ease of Transition						
	Raw Score		0	0	0	1	1
	Weighted Score	100%	0	0	0	1	1
20	Maximize Resource Potential						
	Raw Score		0	-1	-1	2	2
	Weighted Score	100%	0	-1	-1	2	2
21	Timing						
	Raw Score		1	0	0	1	1
	Weighted Score	100%	1	0	0	1	1

Evaluation Criteria	Weighting of Importance	BENEFIT IN IMPLEMENTING GOAL				
		Alternate No. 1	Alternate No. 2	Alternate No. 3	Alternate No. 4	Alternate No. 5
		Lagoon Aeration	Activated Sludge	Oxidation Ditch	Sequencing Batch Reactor (SBR)	Membrane Treatment (MR)
22 Site/Location Considerations						
Raw Score		1	0	0	1	1
Weighted Score	100%	1	0	0	1	1
23 Operator/Public Safety						
Raw Score		-1	0	0	1	2
Weighted Score	100%	-1	0	0	1	2
Raw Score Total		2	-1	0	21	22
Weighted Score Total		4	-5	4	19	20

Legend
2 = Much Better Than Baseline
1 = Better Than Baseline
0 = Same As Baseline (Activated Sludge)
-1 = Worse Than Baseline
-2 = Much Worse Than Baseline

6.1.4 Precision of Estimate

In this chapter, the costs for implementation of the recommended improvements are estimated. As a first step in estimating costs, it is necessary to determine the precision required for the costs based on the amount of detailed information available at the time of the estimate. The two levels used for estimating purposes are preliminary costs, and detailed costs.

6.1.4.1 Preliminary Costs

Preliminary estimates are generally developed early in the formulation of potential projects to aid in determining the feasibility of a project and analyze multiple alternatives. They are also used by the engineer or owner to develop financing for the construction of the project, and budget for increased manpower as well as operation, maintenance, and replacement (OM&R) annual costs. The two levels of approximate costs developed for these purposes are as follows:

Budget Cost Estimate

The budget level cost estimate is the most preliminary of cost estimates, developed once basic concepts and preliminary layouts are prepared. This level groups many cost elements together, to determine the general cost guidelines that will be associated with implementing such a project. Sometimes, the long-term costs associated with operation, maintenance and repair are included, to the extent that they may differ between alternatives being considered. Whereas individual elements will vary widely within the total project cost, the goal of the budget estimate is to provide costs in the range of plus or minus 20 to 30 percent of the expected costs.

Conceptual Cost Estimates

At this level, preliminary sketches or concept layouts of the project have been completed. The major elements of the project have been identified but not yet detailed. Design has been completed only to the level of layout of preferred alternatives, with the details remaining to be completed. The concept is firmed up somewhat from the budget estimate since the major project elements have been scoped and identified, but can still be expected to deviate from actual costs by as much as a plus or minus 15 to 25 percent.

6.1.4.2 Planning Level Cost Estimates

The cost estimates prepared in this Sanitary Sewer System Facilities Plan, should be considered Budget Level Cost Estimates. Basic concepts and preliminary layouts have been established. However, this has been completed with general and in most cases, a

broad understanding of the technical factors and specific site conditions that will affect final design and construction. For larger projects, preliminary design and analysis is warranted to provide further detail and refinement of specific project elements and associated costs.

6.1.5 Basis of Cost Estimates

All costs presented herein are based on current construction practices and recent general bid pricing for similar work. Because these costs will tend to escalate over time, they are presented in year 2015 dollars, and should be adjusted to account for inflation over time if the construction is delayed. The recommended method is to utilize a construction cost index prepared by Engineer News-Record (ENR index), which compiles data from 20 major U.S. cities into an index that is adjusted and published weekly.

The ENR construction cost index for this plan was selected to be 9,992 as published in Engineering News-Record in April 2015. The preliminary design report estimates can be adjusted for future projects by applying the ratio of the ENR index at the time of construction to 9,992, and multiplying the estimated project cost by the result. Although the ENR index is not an absolute measure of trends to a particular area, it is widely recognized as a viable method for estimating the general escalation of construction costs over time.

The costs included in this Facilities Plan reflect current research and contact with equipment suppliers and industry representatives.

In this report, we have grouped costs to represent total construction costs on a “per foot complete” basis for the construction of pipelines. Included in these costs are pipe materials, excavation, native surface restoration (where applicable), an allowance for connection to existing facilities, abandoning existing facilities which are no longer needed, and other miscellaneous items and costs. Since many of these items are not broken out separately, the per foot costs may appear higher than actual construction per foot costs.

6.1.5.1 Engineering, Administration, Legal, and Contingency Costs

For each project that is undertaken, certain costs will be realized that must be figured into the overall cost estimate. History has shown that these costs can be expressed as a percentage of the construction cost with relative accuracy for this level of estimate. The factors we have included in our estimates are summarized in Table 6.2.

Table 6.2 Cost Estimate Factors

Description	Estimated Percentage of Construction Cost
Engineering and Project Administration, includes:	17%
Pre-Design Report, Facilities Plan, Recycled Water Plan	
Geotechnical Investigations	
Design and Construction Surveys	
Design Documents (Plans, Specifications and Contract Documents)	
Construction Inspection	
Record Drawings	
Operation and Maintenance Manuals	
Construction and Financial Management	
Legal, Administrative, Property Acquisition	7%
Contingency:	30%
TOTAL	**54%**

The total engineering, project administration, legal, and contingency factor of 54-percent is applied to each of the project construction cost estimates that follow, yielding the following formula:

Total Estimated Project Cost = 1.54 x Estimated Construction Cost

6.2 Estimated Costs for Capital, and Operation, Maintenance & Replacement for Improvements

Cost estimates for the sewage treatment and disposal system improvements necessary to serve the projected future growth as identified in Sections 4 and 5 are summarized in Table 6.3. Detailed estimates for all improvements are provided in Appendix G. The costs represent both initial capital expenditures, and on-going operations, maintenance and replacement costs (OM&R).

Table 6.3 Summary of Estimated Costs for Alternatives

PROJECT		WASTEWATER TREATMENT IMPROVEMENTS	IRRIGATION / REUSE IMPROVEMENTS	TOTAL
CAPITAL COST - TREATMENTAND REUSE IMPROVEMENTS				
T-1	ALTERNATIVE NO. 1 -LAGOON AERATION RETROFIT	$10,409,553.00	$3,244,472	$13,654,025
T-2	ALTERNATIVE NO. 2 - ACTIVATED SLUDGE	$13,869,410.94	$3,244,472	$17,113,883
T-3	ALTERNATIVE NO. 3 - OXIDATION DITCH	$13,023,125.50	$3,244,472	$16,267,598
T-4	ALTERNATIVE NO. 4 - SEQUENCING BATCH REACTOR (SBR)	$12,240,613.00	$3,244,472	$15,485,085
T-5	ALTERNATIVE NO. 5 - MEMBRANE BIOREACTOR (MBR)	$11,091,927.00	$3,244,472	$14,336,399
ANNUAL OPERATING COST - TREATMENTAND REUSE IMPROVEMENTS				
O-1	ALTERNATIVE NO. 1 -LAGOON AERATION RETROFIT	$476,620	$28,405	$505,025.00
O-2	ALTERNATIVE NO. 2 - ACTIVATED SLUDGE	$471,775	$28,405	$500,180.00
O-3	ALTERNATIVE NO. 3 - OXIDATION DITCH	$479,635	$28,405	$508,040.00
O-4	ALTERNATIVE NO. 4 - SEQUENCING BATCH REACTOR (SBR)	$446,770	$28,405	$475,175.00
O-5	ALTERNATIVE NO. 5 - MEMBRANE BIOREACTOR (MBR)	$464,360	$28,405	$492,765.00
PRESENT WORTH COST - TREATMENTAND REUSE IMPROVEMENTS				
PW-1	ALTERNATIVE NO. 1 -LAGOON AERATION RETROFIT	$16,886,819	$3,630,496	$20,517,315
PW-2	ALTERNATIVE NO. 2 - ACTIVATED SLUDGE	$20,280,833	$3,630,496	$23,911,329
PW-3	ALTERNATIVE NO. 3 - OXIDATION DITCH	$19,541,365	$3,630,496	$23,171,861
PW-4	ALTERNATIVE NO. 4 - SEQUENCING BATCH REACTOR (SBR)	$18,312,217	$3,630,496	$21,942,713
PW-5	ALTERNATIVE NO. 5 - MEMBRANE BIOREACTOR (MBR)	$17,402,579	$3,630,496	$21,033,075

(1) Irrigation/Reuse Improvements assume irrigation at Lafayette Farms.

(2) Present Worth assumes 20-year period with O&M, R Costs at an annual discount rate of 4%.

6.3 Recommended Wastewater Treatment Alternative

6.3.1 Recommended Wastewater Treatment Alternative

Chapter 5 identified several wastewater treatment technologies and effluent reuse alternatives. The associated capital and annual operating and maintenance costs were developed to allow for a cost comparison.

Detailed advantages and disadvantages for both approaches were provided in Chapter 5. Of the five option treatment approaches Option No. 5 (MBR Treatment) offers significant advantages over other technologies including the following:

- When selecting an approach to provide long term wastewater management the flexibility offered by Class A water recycling provided by an MBR, is a significant benefit that will allow the City to adapt to a wider range of potential operational and reuse scenarios.

- Increased flexibility and treatment efficiency also benefits the City's need to meet future permit requirements and potential changes and tightening standards.

- MBR technology is rapidly evolving and already offers much higher treatment efficiencies than can be achieved through lagoon based treatment. It is likely that this trend will continue with the result being increasingly clean effluent.

- The capital cost of MBR equipment and filters is decreasing and the "cost per gallon" for future phases is likely to be less than the initial phase.

- The small foot print of the MBR treatment plant will become increasingly more important as the City continues to grow. Long term the City can expect to decrease the area of the lagoons and consequently increase the buffers around the perimeter.

- It is conceivable that eventually some of the area currently occupied by the lagoons could be converted to industrial or commercially zoned land providing the City with a financial return on the property sale, and a corresponding increase in property tax revenue.

- Producing Class A effluent will allow for a wide range of water recycling opportunities within the City including irrigation of public spaces and private properties.

6.3.2 Description of Recommended Approach

Figure 6-1 provides a schematic and site plan for the recommended approach incorporating revised or new wastewater treatment and effluent reuse facilities. An MBR plant will be constructed in parallel with the existing lagoon system. The MBR will allow for operation in parallel, simultaneously with the facultative lagoons, or separately with the lagoons "off line". Treated effluent from either treatment train can be chlorinated/dechlorinated before discharge. Winter time discharge will be to the existing river outfall. Management of dry weather flows will consist of an approach that combines spray irrigation to agricultural land north of town and continued storage in the lagoons. The new treatment plant would be sized to provide an additional treatment of 0.75 mgd in two phases (0.50 mgd-Phase II, 0.25 mgd-Phase III).

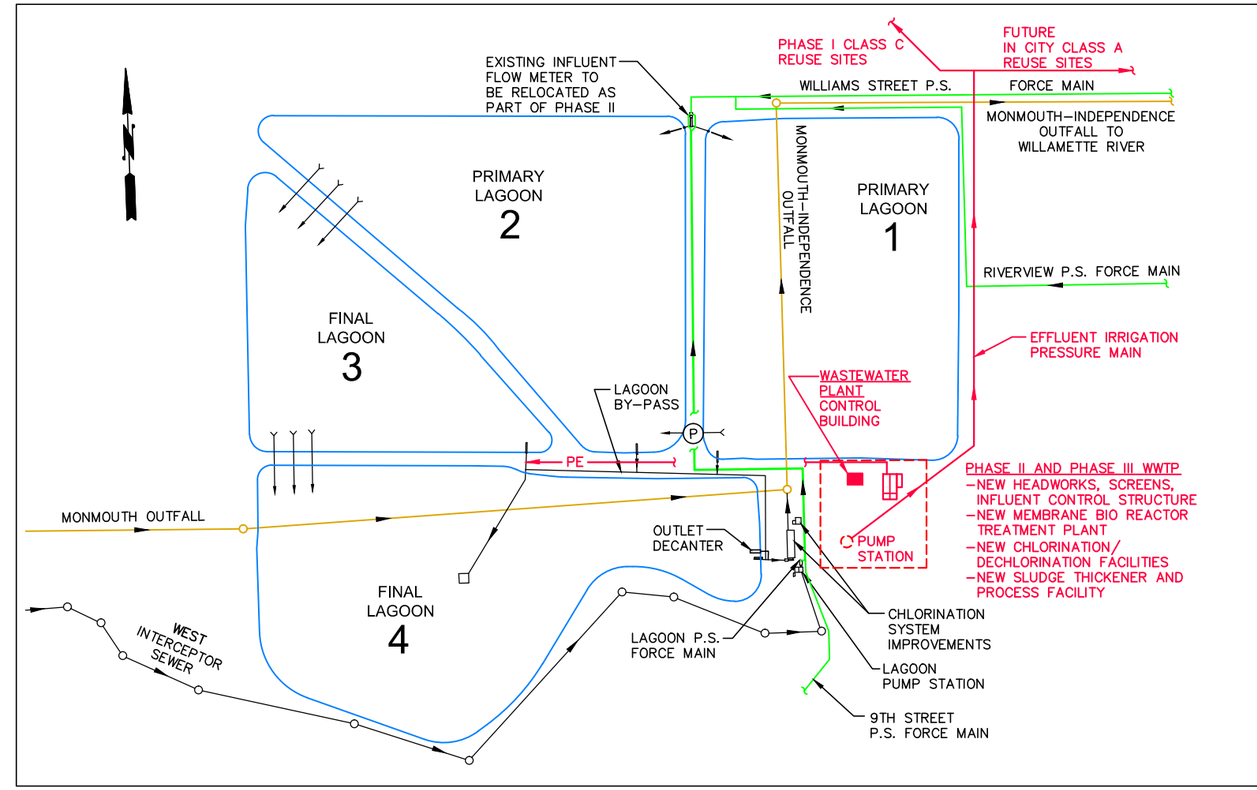
Planning level cost estimates to complete this work are provided in Table 6-4. Chapter 7 provides a more detailed description of the proposed phasing plan.

Table 6.4 Preferred Alternative - Membrane Bioreactor (MBR)/Effluent Reuse (Irrigation)

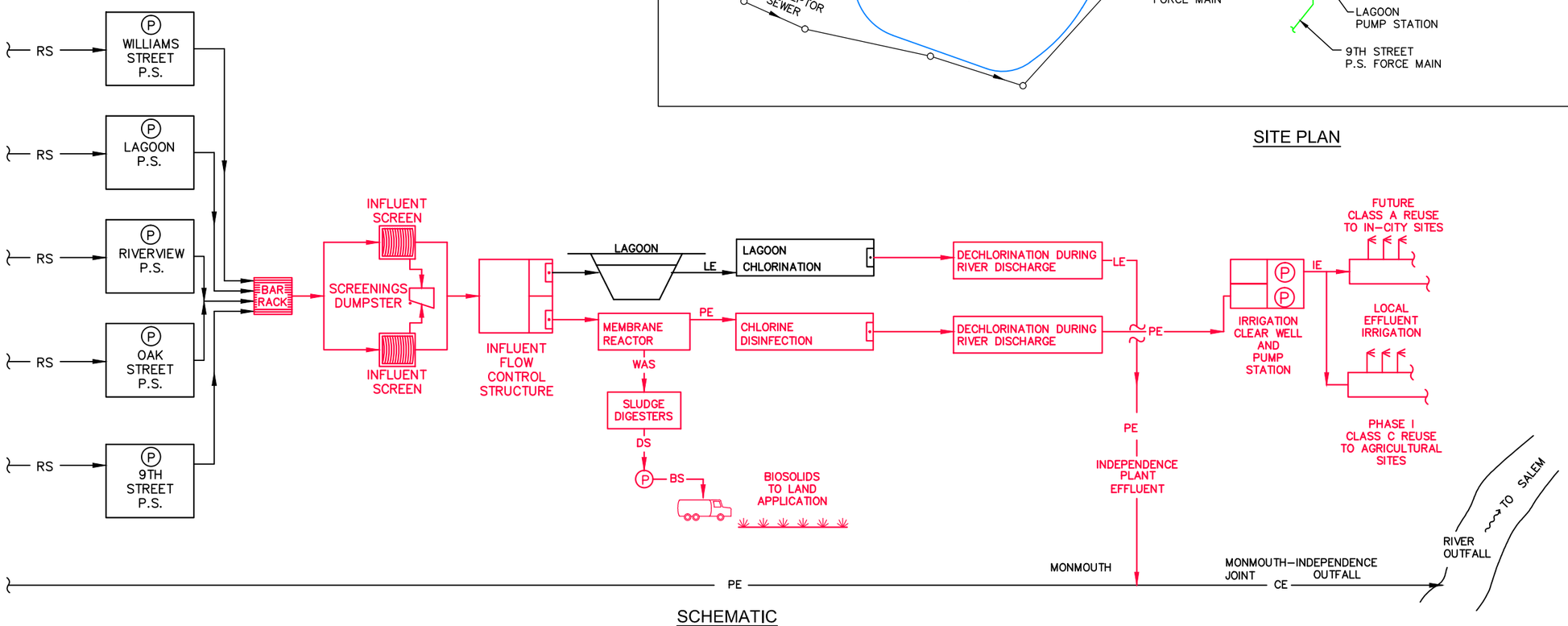
Summary of MBR Improvements – Phases II and III		
Item	**Description**	**Cost**
1	Headworks	$525,900
2	Process	$4,364,300
3	Operation and Control	$566,900
4	Effluent Management	$479,700
5	Piping Reconfiguration	$48,250
6	Solids Management	$1,217,500
	Total Construction Cost **Soft Costs and Contingency @ 54%**	$7,202,550 $3,889,377
	Total Cost	$11,091,927

LEGEND

CE	CHLORINATED EFFLUENT
CGV	CHLORINE GAS VACUUM
CS	CHLORINE SOLUTION
DS	DIGESTED SLUDGE
F	FLOW METER
FM	FORCE MAIN
IE	IRRIGATION EFFLUENT
LE	LAGOON EFFLUENT
(P)	PUMP
PE	PLANT EFFLUENT
RS	RAW SEWAGE
WAS	WASTE SLUDGE



SITE PLAN



SCHEMATIC

CITY OF INDEPENDENCE
POLK COUNTY, OREGON

RECOMMENDED PLAN
MBR/CLASS A-C/REUSE

HISTORIC INDEPENDENCE

FIGURE 6.1

7 RECOMMENDED ALTERNATIVE

7.1 Project Description and Phasing Plan

Chapters 5 and 6 identified several treatment technologies that are appropriate and can provide excellent treatment and management of wastewater during the next 20 years. Preliminary budget cost estimates for capital improvements and annual operation and maintenance were developed to allow for an initial financial comparison. Additional operational and permit considerations were identified along with qualitative comparison of issues and concerns typically associated with wastewater management.

In addition to the treatment of wastewater, an efficient and comprehensive method of managing effluent is required. Alternatives for the management and/or reuse of treated and disinfected effluent were identified in Chapter 5 and associated budget level cost estimates developed in Chapter 6.

Identifying the most cost effective, efficient and appropriate approach to managing wastewater during the next 20-years and beyond requires evaluating a systems approach that considers both treatment and effluent management. The recommended option of constructing a membrane bioreactor (MBR) treatment plant in parallel to the existing lagoon based treatment facility offers significant short term and long term advantages. As part of this capital improvement program the City should develop a recycled effluent reuse program. The recycled effluent program will become an integral part of the City's sewerage system and land use development policy. For this reason the City is in the process of developing a *Recycled Water Use Plan* for review and approval by DEQ. This plan will refine an approach and total area required for reuse sites, and begin the process of identifying legal agreements that will be required.

The improvements should be constructed in Phases.

7.1.1 2017 Capital Improvements – Phase I

The initial phase will focus on development of effluent reuse facilities to allow for recycling of treated and disinfected wastewater through spray irrigation on agricultural properties near the City's UGB. This will address the most immediate capacity concern with wastewater management, the storage of dry weather flows.

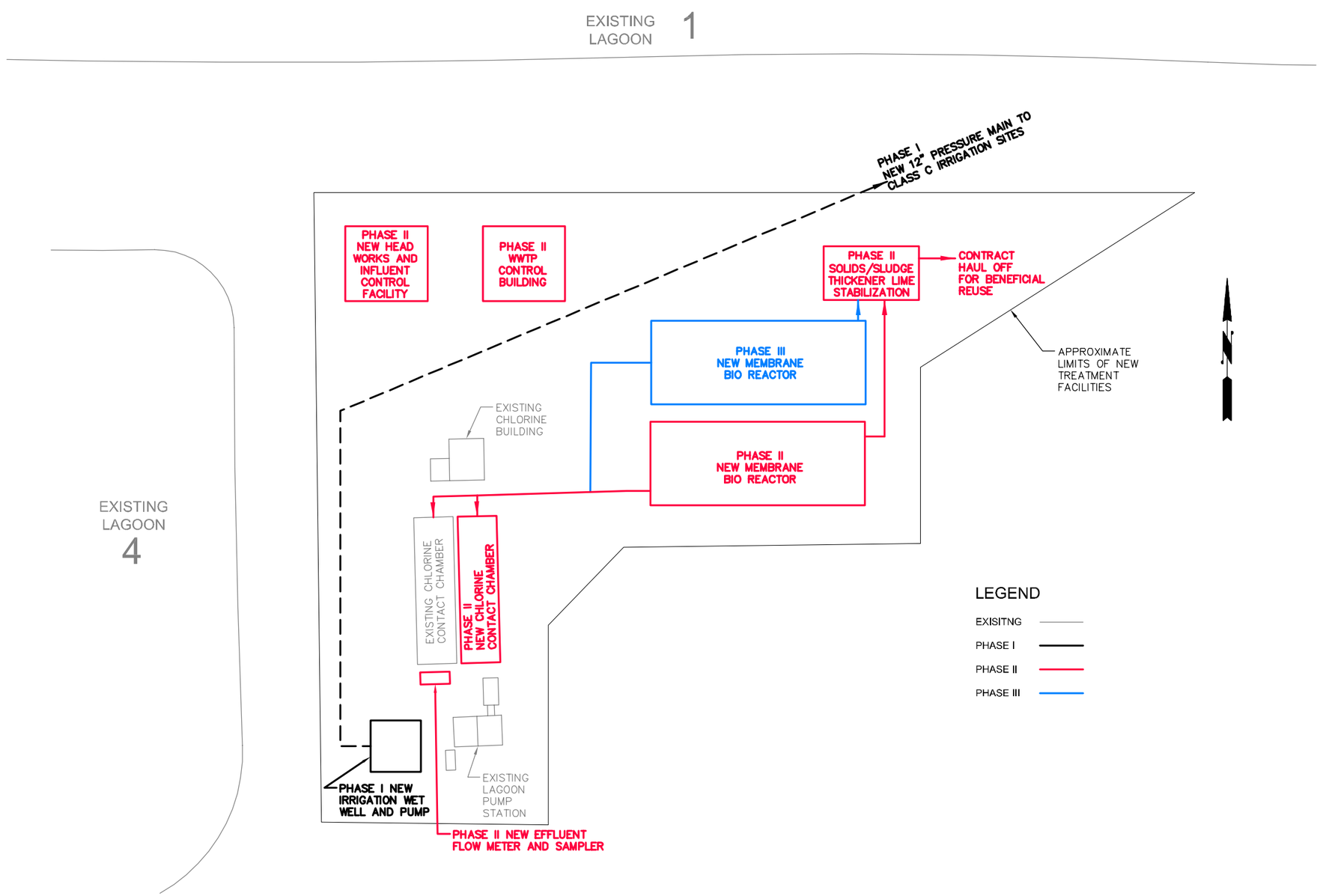
7.1.2 2020 Capital improvements – Phase II

The second phase will address the future need for additional biological treatment associated with population growth through the year 2030. Capital improvements associated with this second phase are recommended to begin construction during the year 2020.

7.1.3 2030 Capital improvements – Phase III

The third phase will focus on the development of additional treatment and effluent reuse capacity to accommodate growth anticipated between the years 2030 and 2035. Capital improvements associated with this third phase are anticipated to be required and begin construction during the year 2029.

Figure 7.1 provides a schematic plan view of major project components and recommended phasing.



CITY OF INDEPENDENCE
POLK COUNTY, OREGON

RECOMMENDED IMPROVEMENTS AND PHASING PLAN

HISTORIC INDEPENDENCE

FIGURE 7.1

7.2 Phase II & III Influent Pumping and Headworks

7.2.1 Influent Pumping and Controls

All sewage is pumped to the plant. All existing pump stations have sufficient lift to discharge to the plant headworks at an estimated elevation of approximately 4 feet above the current discharge elevations, and no additional influent pumps or replacement equipment is required.

The operation and control of lift stations will remain the same. They are constant speed motors cycled on and off automatically depending on the wet well levels in each station. Under the recommended alternative lift station telemetry displays will be available at the plant computer in the proposed Control Building, and in the City Shops Telemetry Room.

7.2.2 Influent Flowmeter

A new Parshall flume and gauge will be constructed at the Headworks structure, just upstream of the Influent Screen. Influent flow, measured by detecting the flume depth with an ultrasonic level sensor, will be monitored and recorded electronically by the plant computer.

7.2.3 Influent Screens

An influent screen performs a huge service to a wastewater plant, removing non-digestable, fixed solids at the front end so they are not run through the liquid process and sludge equipment. The influent screen will also help extend the longevity of the existing lagoons before sludge dewatering and pumping is required.

A 1/8-inch perforated stainless steel influent screen is preferred for membrane bioreactors. The influent screen considered for this plan is the Hycor Heli-Sieve, an in-channel, helical brush-cleaned sewage screen. Other types of self-cleaning influent screens will be compared to the Hycor unit in the preliminary design report. Screens must be evaluated based on cleanability, ability to wash putrescible solids back into the flowstream, warranty, ease of lifting and operator preference. A grinder screen such as the Channel Monster, which macerates the solids and sends them through the process, is not recommended.

7.2.4 Grit Removal

When solids are pumped in a wastewater plant, providing grit removal in the headworks is a service to the operator and owner. The grit removal unit considered under this plan is a single, 8-ft-diameter Smith & Loveless Pista, provided in a cast-in-place portion of the Headworks structure. This unit would be provided with a grit pump, grit classifier/washer, and a grit dumpster.

Aerated grit basins are advertised as requiring a smaller footprint, but only some aerated basins work as effectively as the Pista. The predesign report will compare the Pista against the better aerated grit basin and, depending on the screen provided, against the option of not providing grit removal at all.

7.3 Phase II & III Secondary and Tertiary Treatment Plant

7.3.1 Membrane Bioreactor

Biological treatment will be provided in the membrane bioreactor, consisting of submerged membrane units (SMUs) in a cell within a large concrete basin. The membrane units considered by this plan are flat plate type. The MBR manufacturer would provide the SMUs, permeate pumps, RAS/WAS pumps, blowers and control system.

Blowers, air diffusers and anoxic mixers would be made by manufacturers identified in the predesign. The recommended blowers are variable speed, positive displacement rotary blowers. Blowers would be ramped down and cycled off periodically to minimize energy use. Diffusers will be fine or coarse bubble type.

7.4 Upgrade/Upkeep of Existing Lagoons

The lagoons, which will provide over half of the City's 2035 capacity, must be maintained and kept in good condition.

7.5 Phases I – III Disinfection and Disposal

7.5.1 Effluent Irrigation

The City's commitment to recycled effluent makes chlorination the disinfection system of choice for lagoon effluent. Where UV disinfection does not leave a bactericidal "residual" in pipelines, chlorine does. This makes compliance with the DEQ bacteria limits much easier.

Since the lagoon portion of the plant effluent must be chlorinated, existing chlorine facilities will be improved, and it is cheapest to also disinfect the MBR treatment plant effluent with chlorine. The same consideration applies for dechlorination; if dechlorination is required in a future permit reissuance. If required, a dechlorination system should be sized to accommodate both the lagoon effluent and the treatment plant effluent.

UV disinfection of the treatment plant effluent is possible for MBR effluent and would avoid the need for additional dechlorination facilities, but it is not recommended. Since the treatment plant can produce Class A reuse effluent in the summer, chlorination is required because UV does not provide the needed residual disinfection. Since the plant uses chlorine in the summer, it should also use chlorine in the winter when discharge is to the river.

7.5.2 River Discharge

Treatment plant effluent will be discharged seasonally to the river. The higher quality effluent from the MBR will be blended with lagoon effluent to reduce overall BOD and solids loads.

7.6 Solids Management – Phase II and III

7.6.1 Aerobic Sludge Digester and Storage Tank

Aerobic sludge digestion is recommended as the sludge digestion method. The plant flow and waste load is too small to warrant building an anaerobic digester, an ATAD digester, or for attempting to make and market Class A biosolids. Many small Oregon plants in the same situation use aerobic digestion.

In the aerobic digestion process, waste sludge from the treatment plant is mixed and aerated in an open concrete tank for a period of 15 days or more. Periodically, the aeration is stopped, and the sludge is allowed to settle and thicken, while relatively clear supernatant is withdrawn and returned to the wastewater process. The digested sludge is then transferred to an aerated sludge storage tank, where it is further digested and stored until summertime, when it is stabilized and hauled to approved agricultural fields for disposal.

Under the recommended plan in 2020, a sludge digester and sludge storage tank (estimated to be 80,000 gallons each) will be constructed as part of the membrane bioreactor structure, with a common wall separating the liquid and solids processes. The MBR waste sludge pump may be configured to also function as a sludge transfer pump; or a separate sludge transfer pump will be provided.

In 2030, when more sludge treatment and storage volume is needed, a new MBR tank won't be needed, because the 2020 MBR tank will be fitted with additional membrane units. At that time, a separate, two-cell sludge digester and storage tank will be constructed.

7.6.2 Lime Stabilization and Sludge Hauling

Digested sludge, typically at a density of 2 to 3 percent, will be mixed with hydrated lime to raise the pH for sufficient time to "lime-stabilize" the sludge to remove pathogens and vectors. The lime stabilized sludge is pumped from the storage tank to a 2,500-gallon sludge haul truck, generally provided on contract from a larger City or a private hauler, and trucked to a DEQ-permitted sludge disposal field.

7.6.3 Land Disposal of Digested Sludge

The available digested sludge land application sites will be evaluated during the preliminary design report. Sites are closely regulated by DEQ. As specified in Schedule C of the City's NPDES permit, the City is required to submit a biosolids management plan to DEQ which considers the site characteristics. Frequently, several cities share a biosolids disposal site.

8 FINANCING STRATEGY

8.1 Sources of Funding

There are several potential sources of funding for the capital improvements outlined in this Facilities Plan. Funding sources include but are not limited to user fees, systems development fees (SDC's), grants and loans. The City has successfully utilized all of these funding sources and financial tools to finance previous capital improvement projects. Programs previously used or ones that appear to be attractive at the time this plan was prepared are summarized below:

Oregon Department of Environmental Quality

DEQ administers the Clean Water State Revolving Fund (CWSRF), which provides low-interest loans to public utilities for preparing planning and environmental review documents, design and construction of wastewater facilities and other water quality improvement projects.

Oregon Business Development Department Infrastructure Finance Authority

OBDD-IFA administers the federal Housing and Urban Development Community Development Block Grant (HUD-CDBG) program for "non-entitlement area" within Oregon, as well as the Oregon Lottery-funded Water/Wastewater Financing and Special public Works Fund programs. These programs consist of both grants and loans, and can finance preparation of planning and environmental review documents, in addition to design and construction of public wastewater systems.

United States Department of Agriculture Rural Development

The Rural Development (RD) program was formerly known as the Farmer's Home Administration administers grants and loans focused on constructing and upgrading needed public and private non-profit utility systems, including wastewater systems in small rural communities (population less than 10,000). The City applied for and received over $2.5 million in low interest loans from the Rural Development agency as part of the 1998 sewer improvement project. The terms of the loan originally included a 40-year payback time frame.

Rural Community Assistance Corporation

The RCAC is a private nonprofit organization that provides training and technical assistance with funding through the national Rural Community Assistance Partnership. RCAC is designated a Community Development Financial Institution by the U.S. Department of Treasury and can provide low-interest loans for projects. RCAC

financing can cover feasibility and pre-development expenses to meet USDA-Rural Development's requirements.

Systems Development Charges

Systems development charges (SDC's) are assessed to new development to reflect the impact on existing facilities. Typically this is calculated in terms of hydraulic and biological treatment capacity that must be made available to serve the new development. SDC's must be applied in a manner that requires new development to pay their "fair share" of the cost a

nd impact to the City's sewerage system. This Facilities Plan is an important step in updating a strategy and associated costs to serve the City Limits and Urban Growth Boundary. A fair and equitable distribution of the costs associated with providing service to the entire service area must be determined through the subsequent development of a SDC Methodology. SDC's are an important component of the funding strategy for many capital improvement projects.

User Fees

User fees are applied to cover operational and maintenance costs, and the cost to finance capital improvements. City records indicate that there are 2,285 connections to the sewerage system. The City's recently increased the base monthly sewer user charge is $42.93/month/connection.

Low Interest Loans and Bonds

Low interest loans and bonds are two financial tools that can be used to pay for capital improvements. Both are essentially loans that must be paid back through user fees.

8.2 Financing Plan

The following sections outline the general financial planning timeline for improvements that have been identified as necessary to address current operational capacity concerns, current or future regulatory requirements, and anticipated future capacity short falls. Additional improvements have been identified that are anticipated to be necessary to expand the service area to accommodate projected growth and development within the current City Limits and Urban Growth Boundary.

8.2.1 Introduction

Recommended improvements will be constructed in phases to meet needs associated with future growth. The proposed improvements all represent significant capital improvements with associated costs that are large enough to require grants, loans and/or bonds.

8.2.2 Financial History

8.2.2.1 Cash Flows from Operating Activities

The current user rate is $42.93/month per connection. With 2,285 connections during a normal month the City collects an estimated $98,095/month ($1,177,000/year).

The City recently refinanced approximately $2,750,000 of debt for SRF and USDA bonds for previous sewerage system capital improvement projects with a projected payoff in 2040.

8.2.2.2 Cash Flows from Capital and Related Activities

There are no net positive cash flows into the sewer fund from capital and related activities.

8.2.2.3 Cash and Investments

There are no cash or investments producing positive cash flow to the sewer fund.

8.2.3 Financial Forecast

8.2.3.1 Phase I – Financial Forecast

The spray irrigation improvements proposed under Phase I have an estimated total capital cost of $3,244,472. With an assumed 20 year loan or bond at an annual 4% interest rate, and an increase in total connections to 2,415 the increase to the base monthly User Charge is estimated to be $8.15/month

8.2.3.2 Phase II – Financial Forecast

The wastewater treatment improvements proposed under Phase II have an estimated total capital cost of $6,849,920. It is fair to assume that these improvements would be constructed in response to increased population and growth within the City. Using the 2.5% projected population growth rate and assuming a proportional increase in sewer connections totaling 2,623, the increase to the base monthly User Charge is estimated to be an additional $15.90/month. This assumes a 20 year loan or bond at an annual 4% interest rate.

8.2.3.3 Phase III – Financial Forecast

The wastewater treatment improvements proposed under Phase III have an estimated total capital cost of $4,242,007. It is fair to assume that these improvements would be constructed in response to increased population and growth within the City. Using the 2.5% projected population growth rate and assuming a proportional increase in sewer connections totaling 3,458 the increase to the base monthly User Charge is estimated to

be an additional $7.45/month. This assumes a 20 year loan or bond at an annual 4% interest rate.

9 REFERENCES

Planning Documents and Investigations

1996 City of Independence Sanitary Preliminary Design Report, November 1996

1996 Mixing Zone Study for the Wastewater Treatment Facility Outfall, August 1996

Draft Monmouth-Independence Joint Effluent Reclamation Project Feasibility Study, September 2004

Smoke Testing Reports, 2001, C-More Pipes

Television Inspection Reports, 2001, C-More Pipes,

1994 City of Independence Facilities Plan Update, Volume I-II, August 1994

State of Oregon Department of Environmental Quality Guidelines for Making Flow Projections for Sewage Treatment: MMDWF, MMWWF, PIF

Construction Documents

1978 City of Independence, Oregon Wastewater Treatment Expansion Drawings

1979 Monmouth-Independence, Oregon Outfall Sewer Drawings

1998 Plans for Construction of the Sanitary Sewer System Improvements for the City of Independence, Oregon

Professional Standards of Practice

Design of Municipal Wastewater Treatment Plant, Manual of Practice 8, WEF/ASCE, 1998

Natural Systems for Wastewater Treatment Manual of Practice, FD-16, WEF/ASCE, 1990

APPENDIX A

Existing City of Independence WWTF NPDES Permit

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

Department of Environmental Quality
Western Region – Salem Office
750 Front Street NE, Suite 120, Salem, OR 97301-1039
Telephone: (503) 378-8240

Issued pursuant to ORS 468B.050 and The Federal Clean Water Act

ISSUED TO:
Independence, City of
PO Box 7
Independence, OR 97351

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Treated Wastewater	001	R.M. 95.5

FACILITY TYPE AND LOCATION:
Stabilization Lagoons without Aeration
Independence STP
End of Williams Street
Independence, Oregon

Treatment System Class: I
Collection System Class: Level II

RECEIVING STREAM INFORMATION:
Basin: Willamette
Sub-Basin: Middle Willamette

Receiving Stream: Willamette River
Hydro Code: 1227618456580 95.5 D
County: Polk

EPA REFERENCE NO: OR002044-3

Issued in response to Application No. 990232 received May 28, 1999.
This permit is issued based on the land use findings in the permit record.

Michael H. Kortenhof, Water Quality Manager
Western Region

May 4, 2004
Date

PERMITTED ACTIVITIES

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

Page
Schedule A - Waste Discharge Limitations not to be Exceeded 2
Schedule B - Minimum Monitoring and Reporting Requirements 4
Schedule C - Compliance Conditions and Schedules 7
Schedule D - Special Conditions 8
Schedule F - General Conditions 10

Unless specifically authorized by this permit, by another NPDES or WPCF permit, or by Oregon Administrative Rule, any other direct or indirect discharge to waters of the state is prohibited, including discharge to an underground injection control system.

SCHEDULE A

1. **Waste Discharge Limitations not to be exceeded after permit issuance.**

a. Treated Effluent Outfall 001 (See Note 3)

(1) June 1 - October 31: No discharge to waters of the State (unless approved in writing by the Department)

(2) November 1 - May 31:

Parameter	Average Effluent Concentrations Monthly	Weekly	Monthly* Average lb/day	Weekly* Average lb/day	Daily* Maximum lbs
BOD_5	30 mg/L	45 mg/L	500	750	1000
TSS	50 mg/L	80 mg/L	830	1200	1700

* Average dry weather design flow to the facility equals 0.6 MGD. Winter mass load limits based upon average wet weather design flow to the facility equaling 2.0 MGD.

(3)

Other parameters	Limitations
E. coli Bacteria	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL. (See Note 1)
pH	Shall be within the range of 6.0 - 9.0
BOD_5 and TSS Removal Efficiency	Shall not be less than 85% monthly average for BOD_5 and 65% monthly for TSS.
Total Chlorine Residual	Shall not exceed 1.0 mg/l daily maximum.

(4) Except as provided for in OAR 340-45-080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-41 except in the following defined mixing zone:

The allowable mixing zone is that portion of the Willamette River contained within a band extending out fifty (50) feet from the west bank of the river and extending from a point fifty (50) feet upstream of the outfall to a point three-hundred (300) feet downstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within thirty (30) feet of the point of discharge.

(5) 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 21 and June 1, and if the permittee demonstrates to the Department's satisfaction that no increase 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.

(6) No activities shall be conducted that could cause an adverse impact on existing or potential beneficial uses of groundwater. All wastewater and process related residuals shall be

managed and disposed in a manner that will prevent a violation of the Groundwater Quality Protection Rules (OAR 340-040).

NOTES:

1. If a single sample exceeds 406 organisms per 100 mL, then five consecutive re-samples may be taken at four-hour intervals beginning within 48 hours after the original sample was taken. If the log mean of the five re-samples is less than or equal to 126 organisms per 100 mL, a violation shall not be triggered.

2. Upon approval of a Total Maximum Daily Load for this sub-basin for any water quality limited parameter, this permit may be re-opened to establish new limits and/or new conditions or requirements in accordance with any Waste Load Allocation (WLA) assigned to this source by the TMDL.

SCHEDULE B

1. **Minimum Monitoring and Reporting Requirements** (unless otherwise approved in writing by the Department).

The permittee shall monitor the parameters as specified below at the locations indicated. The laboratory used by the permittee to analyze samples shall have a quality assurance/quality control (QA/QC) program to verify the accuracy of sample analysis. If QA/QC requirements are not met for any analysis, the results shall be included in the report, but not used in calculations required by this permit. When possible, the permittee shall re-sample in a timely manner for parameters failing the QA/QC requirements, analyze the samples, and report the results.

a. Influent

The facility influent sampling locations are the following:

Influent samples and measurements are taken just after the Parshall Flume and just before flowing by gravity through an open air rocky rip-rap channel to the first lagoon cell.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Annually	Verification
BOD_5	Weekly	Composite
TSS	Weekly	Composite
pH	3/Week	Grab

b. Treated Effluent Outfall 001

The facility effluent sampling locations are the following:

* Effluent flow measurements are taken from just before entering the chlorine contact chamber. Effluent grab samples are collected just after the chlorine contact chamber.

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Annually	Verification
BOD_5	Weekly	Composite
TSS	Weekly	Composite
pH	3/Week	Grab
E. coli	Weekly	Grab (See Note 1)
Quantity Chlorine Used	Daily	Measurement
Chlorine Residual	Daily	Grab
Pounds Discharged (BOD_5 and TSS)	Weekly	Calculation
Average Percent Removed (BOD_5 and TSS)	Monthly	Calculation
Iron	Annually in January (See Note 2)	Grab
Mercury	Annually in January (See Note 3)	Grab
Dissolved Oxygen	Monthly when Discharging (See Note 4)	Grab
Ammonia	Monthly when Discharging (See Note 4)	Grab

c. Temperature Monitoring

Item or Parameter	Minimum Frequency	Type of Sample
Effluent Temperature, Daily Max	2/Week	Grab
Excess Thermal Load October 15 – May 15	Weekly	Calculation (See Note 5)

2. **Reporting Procedures**

a. Monitoring results shall be reported on approved forms. The reporting period is the calendar month. Reports must be submitted to the Department's Western Region - Salem office by the 15th day of the following month.

b. State monitoring reports shall identify the name, certificate classification and grade level of each principal operator designated by the permittee as responsible for supervising the wastewater collection and treatment systems during the reporting period. Monitoring reports shall also identify each system classification as found on page one of this permit.

c. Monitoring reports shall also include a record of the quantity and method of use of all sludge removed from the treatment facility and a record of all applicable equipment breakdowns and bypassing.

3. **Report Submittals**

a. The permittee shall have in place a program to identify and reduce inflow and infiltration into the sewage collection system. An annual report shall be submitted to the Department by February 1 each year which details sewer collection maintenance activities that reduce inflow and infiltration. The report shall state those activities that have been done in the previous year and those activities planned for the following year.

NOTES:

1. *E. coli* monitoring must be conducted according to any of the following test procedures as specified in **Standard Methods for the Examination of Water and Wastewater, 19th Edition**, or according to any test procedure that has been authorized and approved in writing by the Director or an authorized representative:

Method	Reference	Page	Method Number
mTEC agar, MF	Standard Methods, 18th Edition	9-29	9213 D
NA-MUG, MF	Standard Methods, 19th Edition	9-63	9222 G
Chromogenic Substrate, MPN	Standard Methods, 19th Edition	9-65	9223 B
Colilert QT	Idexx Laboratories, Inc.		

2. The permittee shall monitor for iron annually in January for four consecutive sampling events. A data analysis report shall be submitted with the next permit renewal application in accordance with Schedule D, Condition 5 of this permit. Effluent monitoring for iron shall be conducted using a test method with a detection limit of 0.1 mg/L or less.

3. The permittee shall monitor mercury annually in January for two consecutive sampling events. Monitoring for mercury shall be conducted in accordance with EPA Method 1631. After two sampling events, mercury monitoring may be discontinued unless otherwise notified in writing by the Department. A data analysis report shall be submitted with the next permit renewal application in accordance with Schedule D, Condition 5 of this permit.

4. Ammonia and Dissolved Oxygen monitoring will be performed monthly during the discharge season to determine the impact on the dissolved oxygen of the receiving stream. If the Department determines that there is an impact on the dissolved oxygen concentration in the receiving stream, the Department may re-open the permit to include an ammonia limit in Schedule A or other conditions or requirements. Ammonia and Dissolved Oxygen monitoring may be discontinued after one full discharge season unless otherwise notified in writing by the Department. The Department will make its determination based on whether or not the discharge is contributing to the depletion of dissolved oxygen in the receiving stream.

5. The seven day average Excess Thermal Load (ETL) shall be calculated based on the weekly average effluent temperature and flow and the applicable temperature criteria as follows:

June 1- October 31
ETL = average effluent flow (MGD) x (average effluent temperature °C – 13 °C) x 3.785 = Thermal load, in million kcals/day.

SCHEDULE C

Compliance Schedules and Conditions

1. Six (6) months prior to the removal of accumulated solids from the lagoon, the permittee shall submit to the Department a revised biosolids management plan developed in accordance with Oregon Administrative Rule 340, Division 50, "Land Application of Domestic Wastewater Treatment Facility Biosolids, Biosolids Derived Products, and Domestic Septage". Upon approval of the plan by the Department, the plan shall be implemented by the permittee.

2. The permittee is expected to meet the compliance dates which have been established in this schedule. Either prior to or no later than 14 days following any lapsed compliance date, the permittee shall submit to the Department a notice of compliance or noncompliance with the established schedule. The Director may revise a schedule of compliance if he/she determines good and valid cause resulting from events over which the permittee has little or no control.

SCHEDULE D

Special Conditions

1. An adequate contingency plan for prevention and handling of spills and unplanned discharges shall be in force at all times. A continuing program of employee orientation and education shall be maintained to ensure awareness of the necessity of good in-plant control and quick and proper action in the event of a spill or accident.

2. All biosolids shall be managed in accordance with the current, DEQ approved biosolids management plan, and the site authorization letters issued by the DEQ. Any changes in solids management activities that significantly differ from operations specified under the approved plan require the prior written approval from DEQ.

   All new Class B biosolids applications sites shall meet the site selection and nutrient loading criteria set forth in OAR 340-050-0070 and OAR 340-050-0080.

   This permit may be modified to incorporate any applicable standard for biosolids use or disposal promulgated under section 405(d) of the Clean Water Act, if the standard for biosolids use or disposal is more stringent than any requirements for biosolids use or disposal in the permit, or controls a pollutant or practice not limited in this permit.

3. The permittee shall comply with Oregon Administrative Rules (OAR), Chapter 340, Division 49, "Regulations Pertaining To Certification of Wastewater System Operator Personnel" and accordingly:

   a. The permittee shall have its wastewater system supervised by one or more operators who are certified in a classification <u>and</u> grade level (equal to or greater) that corresponds with the classification (collection and/or treatment) of the system to be supervised as specified on page one of this permit.

Note: A "supervisor" is defined as the person exercising authority for establishing and executing the specific practice and procedures of operating the system in accordance with the policies of the permittee and requirements of the waste discharge permit. "Supervise" means responsible for the technical operation of a system, which may affect its performance or the quality of the effluent produced. Supervisors are not required to be on-site at all times.

   b. The permittee's wastewater system may not be without supervision (as required by Special Condition 3.a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified in the proper classification <u>and</u> at grade level I or higher.

   c. The permittee is responsible for ensuring the wastewater system has a properly certified supervisor available at all times to respond on-site at the request of the permittee and to any other operator.

   d. The permittee shall notify the Department of Environmental Quality in writing within thirty (30) days of replacement or re-designation of certified operators responsible for supervising wastewater system operation. The notice shall be filed with the Water Quality Division, Operator Certification Program, 811 SW 6th Ave, Portland, OR 97204. This requirement is in addition to the reporting requirements contained under Schedule B of this permit.

   e. Upon written request, the Department may grant the permittee reasonable time, not to exceed 120 days, to obtain the services of a qualified person to supervise the wastewater system. The written request must include justification for the time needed, a schedule for recruiting and hiring, the date the system supervisor availability ceased and the name of the alternate system supervisor(s) as required by 3.b. above.

4. The permittee shall not be required to perform a hydrogeologic characterization or groundwater monitoring during the term of this permit provided:

   a. The facilities are operated in accordance with the permit conditions, and;

   b. There are no adverse groundwater quality impacts (complaints or other indirect evidence) resulting from the facility's operation.

   If warranted, at permit renewal the Department may evaluate the need for a full assessment of the facilities impact on groundwater quality.

5. The permittee shall monitor for iron annually in January for four consecutive sampling events. The permittee shall monitor for mercury annually in January for two consecutive sampling events. A data analysis report shall be submitted with the next permit renewal application.

6. The permittee shall notify the DEQ Western Region - Salem Office (phone: (503) 378-8240) in accordance with the response times noted in the General Conditions of this permit, of any malfunction so that corrective action can be coordinated between the permittee and the Department.

NPDES GENERAL CONDITIONS
(SCHEDULE F)

SECTION A. STANDARD CONDITIONS

1. Duty to Comply

The permittee must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of Oregon Revised Statutes (ORS) 468B.025 and is grounds for enforcement action; for permit termination, suspension, or modification; or for denial of a permit renewal application.

2. Penalties for Water Pollution and Permit Condition Violations

Oregon Law (ORS 468.140) allows the Director to impose civil penalties up to $10,000 per day for violation of a term, condition, or requirement of a permit.

In addition, a person who unlawfully pollutes water as specified in ORS 468.943 or ORS 468.946 is subject to criminal prosecution.

3. Duty to Mitigate

The permittee shall take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit which has a reasonable likelihood of adversely affecting human health or the environment. In addition, upon request of the Department, the permittee shall correct any adverse impact on the environment or human health resulting from noncompliance with this permit, including such accelerated or additional monitoring as necessary to determine the nature and impact of the noncomplying discharge.

4. Duty to Reapply

If the permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the permittee must apply for and have the permit renewed. The application shall be submitted at least 180 days before the expiration date of this permit.

The Director may grant permission to submit an application less than 180 days in advance but no later than the permit expiration date.

5. Permit Actions

This permit may be modified, suspended, revoked and reissued, or terminated for cause including, but not limited to, the following:

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute;

b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts; or

c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge.

The filing of a request by the permittee for a permit modification or a notification of planned changes or anticipated noncompliance, does not stay any permit condition.

6. Toxic Pollutants

The permittee shall comply with any applicable effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

7. Property Rights

The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege.

8. Permit References

Except for effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants and standards for sewage sludge use or disposal established under Section 405(d) of the Clean Water Act, all rules and statutes referred to in this permit are those in effect on the date this permit is issued.

SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance

The permittee shall at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) which are installed or used by the permittee to achieve compliance with the conditions of this permit. Proper operation and maintenance also includes adequate laboratory controls, and appropriate quality assurance procedures. This provision requires the operation of back-up or auxiliary facilities or similar systems which are installed by a permittee only when the operation is necessary to achieve compliance with the conditions of the permit.

2. Duty to Halt or Reduce Activity

For industrial or commercial facilities, upon reduction, loss, or failure of the treatment facility, the permittee shall, to the extent necessary to maintain compliance with its permit, control production or all discharges or both until the facility is restored or an alternative method of treatment is provided. This requirement applies, for example, when the primary source of power of the treatment facility fails or is reduced or lost. It shall not be a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.

3. Bypass of Treatment Facilities

a. Definitions

(1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The term "bypass" does not include nonuse of singular or multiple units or processes of a treatment works when the nonuse is insignificant to the quality and/or quantity of the effluent produced by the treatment works. The term "bypass" does not apply if the diversion does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation.

(2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities or treatment processes which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.

b. Prohibition of bypass.

(1) Bypass is prohibited unless:

(a) Bypass was necessary to prevent loss of life, personal injury, or severe property damage;

(b) There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgement to prevent a bypass which occurred during normal periods of equipment downtime or preventative maintenance; and

(c) The permittee submitted notices and requests as required under General Condition B.3.c.

(2) The Director may approve an anticipated bypass, after considering its adverse effects and any alternatives to bypassing, when the Director determines that it will meet the three conditions listed above in General Condition B.3.b.(1).

c. Notice and request for bypass.

(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, it shall submit prior written notice, if possible at least ten days before the date of the bypass.

(2) Unanticipated bypass. The permittee shall submit notice of an unanticipated bypass as required in General Condition D.5.

4. Upset

a. Definition. "Upset" means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operation error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventative maintenance, or careless or improper operation.

b. Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology based permit effluent limitations if the requirements of General Condition B.4.c are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review.

c. Conditions necessary for a demonstration of upset. A permittee who wishes to establish the affirmative defense of upset shall demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:

(1) An upset occurred and that the permittee can identify the causes(s) of the upset;

(2) The permitted facility was at the time being properly operated;

(3) The permittee submitted notice of the upset as required in General Condition D.5, hereof (24-hour notice); and

(4) The permittee complied with any remedial measures required under General Condition A.3 hereof.

d. Burden of proof. In any enforcement proceeding the permittee seeking to establish the occurrence of an upset has the burden of proof.

5. Treatment of Single Operational Event

For purposes of this permit, A Single Operational Event which leads to simultaneous violations of more than one pollutant parameter shall be treated as a single violation. A single operational event is an exceptional incident which causes simultaneous, unintentional, unknowing (not the result of a knowing act or omission), temporary noncompliance with more than one Clean Water Act effluent discharge pollutant parameter. A single operational event does not include Clean Water Act violations involving discharge without a NPDES permit or noncompliance to the extent caused by improperly designed or inadequate treatment facilities. Each day of a single operational event is a violation.

6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations

a. Definitions

(1) "Overflow" means the diversion and discharge of waste streams from any portion of the wastewater conveyance system including pump stations, through a designed overflow device or structure, other than discharges to the wastewater treatment facility.

(2) "Severe property damage" means substantial physical damage to property, damage to the conveyance system or pump station which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of an overflow.

(3) "Uncontrolled overflow" means the diversion of waste streams other than through a designed overflow device or structure, for example to overflowing manholes or overflowing into residences, commercial establishments, or industries that may be connected to a conveyance system.

b. Prohibition of overflows. Overflows are prohibited unless:

(1) Overflows were unavoidable to prevent an uncontrolled overflow, loss of life, personal injury, or severe property damage;

(2) There were no feasible alternatives to the overflows, such as the use of auxiliary pumping or conveyance systems, or maximization of conveyance system storage; and

(3) The overflows are the result of an upset as defined in General Condition B.4. and meeting all requirements of this condition.

c. Uncontrolled overflows are prohibited where wastewater is likely to escape or be carried into the waters of the State by any means.

d. Reporting required. Unless otherwise specified in writing by the Department, all overflows and uncontrolled overflows must be reported orally to the Department within 24 hours from the time the permittee becomes aware of the overflow. Reporting procedures are described in more detail in General Condition D.5.

7. Public Notification of Effluent Violation or Overflow

If effluent limitations specified in this permit are exceeded or an overflow occurs, upon request by the Department, the permittee shall take such steps as are necessary to alert the public about the extent and nature of the discharge. Such steps may include, but are not limited to, posting of the river at access points and other places, news releases, and paid announcements on radio and television.

8. Removed Substances

Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters shall be disposed of in such a manner as to prevent any pollutant from such materials from entering public waters, causing nuisance conditions, or creating a public health hazard.

SECTION C. MONITORING AND RECORDS

1. Representative Sampling

Sampling and measurements taken as required herein shall be representative of the volume and nature of the monitored discharge. All samples shall be taken at the monitoring points specified in this permit and shall be taken, unless otherwise specified, before the effluent joins or is diluted by any other waste stream, body of water, or substance. Monitoring points shall not be changed without notification to and the approval of the Director.

2. Flow Measurements

Appropriate flow measurement devices and methods consistent with accepted scientific practices shall be selected and used to ensure the accuracy and reliability of measurements of the volume of monitored discharges. The devices shall be installed, calibrated and maintained to insure that the accuracy of the measurements is consistent with the accepted capability of that type of device. Devices selected shall be capable of measuring flows with a maximum deviation of less than ± 10 percent from true discharge rates throughout the range of expected discharge volumes.

3. Monitoring Procedures

Monitoring must be conducted according to test procedures approved under 40 CFR Part 136, unless other test procedures have been specified in this permit.

4. Penalties of Tampering

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate, any monitoring device or method required to be maintained under this permit shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than two years, or by both. If a conviction of a person is for a violation committed after a first conviction of such person, punishment is a fine not more than $20,000 per day of violation, or by imprisonment of not more than four years or both.

5. Reporting of Monitoring Results

Monitoring results shall be summarized each month on a Discharge Monitoring Report form approved by the Department. The reports shall be submitted monthly and are to be mailed, delivered or otherwise transmitted by the 15th day of the following month unless specifically approved otherwise in Schedule B of this permit.

6. Additional Monitoring by the Permittee

If the permittee monitors any pollutant more frequently than required by this permit, using test procedures approved under 40 CFR 136 or as specified in this permit, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the Discharge Monitoring Report. Such increased frequency shall also be indicated. For a pollutant parameter that may be sampled more than once per day (e.g., Total Chlorine Residual), only the average daily value shall be recorded unless otherwise specified in this permit.

7. Averaging of Measurements

Calculations for all limitations which require averaging of measurements shall utilize an arithmetic mean, except for bacteria which shall be averaged as specified in this permit.

8. Retention of Records

Except for records of monitoring information required by this permit related to the permittee's sewage sludge use and disposal activities, which shall be retained for a period of at least five years (or longer as required by 40 CFR part 503), the permittee shall retain records of all monitoring information, including all calibration and maintenance records of all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this permit, and records of all data used to complete the application for this permit, for a period of at least 3 years from the date of the sample, measurement, report or application. This period may be extended by request of the Director at any time.

9. Records Contents

Records of monitoring information shall include:

a. The date, exact place, time and methods of sampling or measurements;

b. The individual(s) who performed the sampling or measurements;

c. The date(s) analyses were performed;

d. The individual(s) who performed the analyses;

e. The analytical techniques or methods used; and

f. The results of such analyses.

10. Inspection and Entry

The permittee shall allow the Director, or an authorized representative upon the presentation of credentials to:

a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;

b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;

c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit, and

d. Sample or monitor at reasonable times, for the purpose of assuring permit compliance or as otherwise authorized by state law, any substances or parameters at any location.

SECTION D. REPORTING REQUIREMENTS

1. Planned Changes

The permittee shall comply with Oregon Administrative Rules (OAR) 340, Division 52, "Review of Plans and Specifications". Except where exempted under OAR 340-52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers shall be commenced until the plans and specifications are submitted to and approved by the Department. The permittee shall give notice to the Department as soon as possible of any planned physical alternations or additions to the permitted facility.

2. Anticipated Noncompliance

The permittee shall give advance notice to the Director of any planned changes in the permitted facility or activity which may result in noncompliance with permit requirements.

3. Transfers

This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and the rules of the Commission. No permit shall be transferred to a third party without prior written approval from the Director. The permittee shall notify the Department when a transfer of property interest takes place.

4. Compliance Schedule

Reports of compliance or noncompliance with, or any progress reports on interim and final requirements contained in any compliance schedule of this permit shall be submitted no later than 14 days following each schedule date. Any reports of noncompliance shall include the cause of noncompliance, any remedial actions taken, and the probability of meeting the next scheduled requirements.

5. Twenty-Four Hour Reporting

The permittee shall report any noncompliance which may endanger health or the environment. Any information shall be provided orally (by telephone) within 24 hours, unless otherwise specified in this permit, from the time the permittee becomes aware of the circumstances. During normal business hours, the Department's Regional office shall be called. Outside of normal business hours, the Department shall be contacted at 1-800-452-0311 (Oregon Emergency Response System).

A written submission shall also be provided within 5 days of the time the permittee becomes aware of the circumstances. If the permittee is establishing an affirmative defense of upset or bypass to any offense under ORS 468.922 to 468.946, and in which case if the original reporting notice was oral, delivered written notice must be made to the Department or other agency with regulatory jurisdiction within 4 (four) calendar days. The written submission shall contain:

a. A description of the noncompliance and its cause;

b. The period of noncompliance, including exact dates and times;

c. The estimated time noncompliance is expected to continue if it has not been corrected;

d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and

e. Public notification steps taken, pursuant to General Condition B.7.

The following shall be included as information which must be reported within 24 hours under this paragraph:

a. Any unanticipated bypass which exceeds any effluent limitation in this permit.

b. Any upset which exceeds any effluent limitation in this permit.

c. Violation of maximum daily discharge limitation for any of the pollutants listed by the Director in this permit.

The Department may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

6. Other Noncompliance

The permittee shall report all instances of noncompliance not reported under General Condition D.4 or D.5, at the time monitoring reports are submitted. The reports shall contain:

a. A description of the noncompliance and its cause;

b. The period of noncompliance, including exact dates and times;

c. The estimated time noncompliance is expected to continue if it has not been corrected; and

d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

7. Duty to Provide Information

The permittee shall furnish to the Department, within a reasonable time, any information which the Department may request to determine compliance with this permit. The permittee shall also furnish to the Department, upon request, copies of records required to be kept by this permit.

Other Information: When the permittee becomes aware that it failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application or any report to the Department, it shall promptly submit such facts or information.

8. Signatory Requirements

All applications, reports or information submitted to the Department shall be signed and certified in accordance with 40 CFR 122.22.

9. Falsification of Information

A person who supplies the Department with false information, or omits material or required information, as specified in ORS 468.953 is subject to criminal prosecution.

10. Changes to Indirect Dischargers - **[Applicable to Publicly Owned Treatment Works (POTW) only]**

The permittee must provide adequate notice to the Department of the following:

a. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of the Clean Water Act if it were directly discharging those pollutants and;

b. Any substantial change in the volume or character of pollutants being introduced into the POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.

c. For the purposes of this paragraph, adequate notice shall include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

11. Changes to Discharges of Toxic Pollutant - **[Applicable to existing manufacturing, commercial, mining, and silvicultural dischargers only]**

The permittee must notify the Department as soon as they know or have reason to believe of the following:

a. That any activity has occurred or will occur which would result in the discharge, on a routine or frequent basis, of any toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels:

(1) One hundred micrograms per liter (100 □g/L);

(2) Two hundred micrograms per liter (200 □g/L) for acrolein and acrylonitrile; five hundred micrograms per liter (500 □g/L) for 2,4-dinitrophenol and for 2-methyl-4,6-dinitrophenol; and one milligram per liter (1 mg/L) for antimony;

(3) Five (5) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

b. That any activity has occurred or will occur which would result in any discharge, on a non-routine or infrequent basis, of a toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels":

(1) Five hundred micrograms per liter (500 □g/L);

(2) One milligram per liter (1 mg/L) for antimony;

(3) Ten (10) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

SECTION E. DEFINITIONS

1. BOD means five-day biochemical oxygen demand.

2. TSS means total suspended solids.

3. mg/L means milligrams per liter.

4. kg means kilograms.

5. m^3/d means cubic meters per day.

6. MGD means million gallons per day.

7. Composite sample means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow.

8. FC means fecal coliform bacteria.

9. Technology based permit effluent limitations means technology-based treatment requirements as defined in 40 CFR 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR 340-41.

10. CBOD means five day carbonaceous biochemical oxygen demand.

11. Grab sample means an individual discrete sample collected over a period of time not to exceed 15 minutes.

12. Quarter means January through March, April through June, July through September, or October through December.

13. Month means calendar month.

14. Week means a calendar week of Sunday through Saturday.

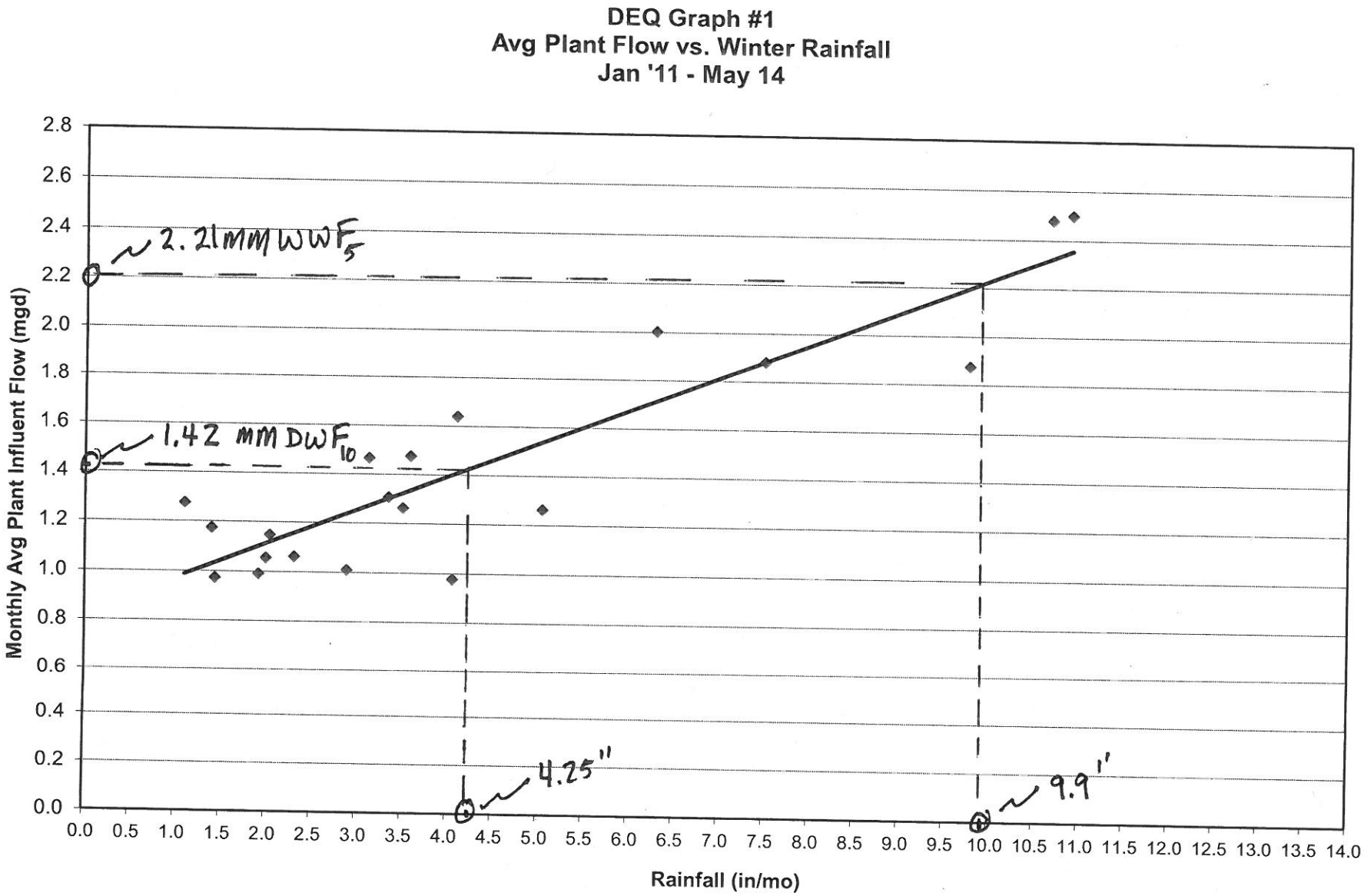
15. Total residual chlorine means combined chlorine forms plus free residual chlorine.

16. The term "bacteria" includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.

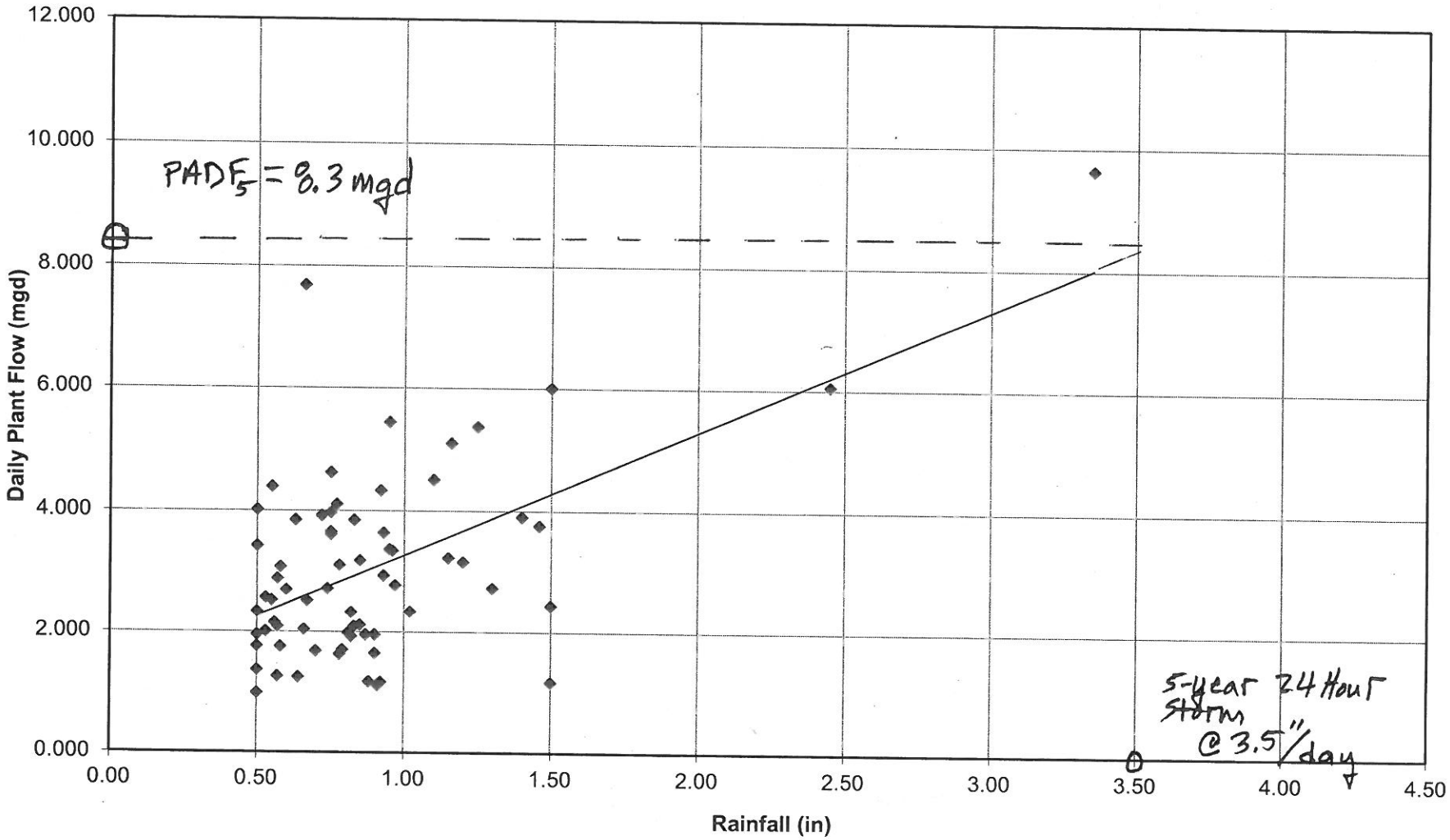
17. POTW means a publicly owned treatment works.

APPENDIX B

System Wide Flow Analysis
DEQ Flow Projections
Historic System Wide Flow Projection


DEQ Graph #1
Avg Plant Flow vs. Winter Rainfall
Jan '11 - May 14
Monthly Avg Plant Influent Flow (mgd)
Rainfall (in/mo)
~2.21 MM WWF5
1.42 MM DWF10
4.25"
9.9"
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0 4.5 5.0 5.5 6.0 6.5 7.0 7.5 8.0 8.5 9.0 9.5 10.0 10.5 11.0 11.5 12.0 12.5 13.0 13.5 14.0

DEQ Graph #2
Daily Plant Flow vs. Daily Rainfall



DEQ GRAPH #2 DATA

DAY MONTH YEAR	PRECIP. (IN)	TOTAL INFLUENT (MGD)	DAY MONTH YEAR	PRECIP. (IN)	TOTAL INFLUENT (MGD)
27-Feb-11	0.50	1.002	10-Mar-11	0.81	1.987
19-Mar-11	0.50	1.775	15-Feb-11	0.82	1.935
29-Jan-12	0.50	1.957	5-Dec-14	0.82	2.330
16-Mar-12	0.50	3.429	24-Jan-12	0.83	3.853
20-Dec-12	0.50	4.016	19-Dec-14	0.83	2.093
26-Dec-12	0.50	2.343	12-Jan-11	0.85	2.116
5-Mar-13	0.50	1.384	1-Mar-12	0.85	3.187
18-Mar-12	0.53	2.010	14-Mar-11	0.87	1.961
6-Dec-14	0.53	2.573	26-Mar-14	0.88	1.190
15-Mar-12	0.55	4.392	19-Mar-13	0.90	1.661
5-Dec-12	0.55	2.529	28-Mar-14	0.90	1.960
27-Mar-14	0.56	2.158	24-Sep-14	0.91	1.129
17-Mar-11	0.57	2.893	23-Apr-11	0.92	1.182
23-Feb-13	0.57	1.277	4-Dec-12	0.92	4.330
6-Apr-13	0.57	2.093	16-Jan-11	0.93	2.941
9-Feb-12	0.58	1.764	28-Dec-11	0.93	3.649
4-Dec-14	0.58	3.082	30-Mar-12	0.95	5.445
20-Mar-12	0.60	2.698	2-Dec-12	0.95	3.376
29-Feb-12	0.63	3.852	16-Dec-12	0.96	3.356
5-Apr-13	0.64	1.265	11-Dec-14	0.97	2.780
19-Jan-12	0.66	7.693	24-Apr-14	1.02	2.341
19-Dec-12	0.66	2.043	12-Feb-14	1.10	4.507
17-Feb-14	0.67	2.526	14-Feb-14	1.15	3.236
16-Jan-12	0.70	1.693	16-Mar-11	1.16	5.105
22-Mar-12	0.72	3.925	12-Mar-12	1.20	3.165
9-Mar-14	0.74	2.720	6-Mar-14	1.25	5.373
20-Jan-12	0.75	4.621	28-Feb-11	1.30	2.733
21-Mar-12	0.75	3.971	29-Mar-12	1.40	3.906
1-Dec-12	0.75	3.620	29-Dec-11	1.46	3.768
19-Feb-14	0.75	3.661	12-Mar-11	1.50	1.182
16-Feb-14	0.77	4.104	13-Mar-11	1.50	2.447
14-Mar-12	0.78	3.114	20-Dec-14	1.50	6.005
5-Mar-14	0.78	1.643	17-Jan-12	2.45	6.056
27-Dec-11	0.79	1.716	18-Jan-12	3.35	9.636
10-Mar-11	0.81	1.987			

GHD

CALCULATIONS

Client	City of Independence	Job no.	12149-10001	Sheet	1 of 1
Project	Sanitary Sewerage Facilities Plan	Calcs by	DMW	Date	4/9/15
Subject	MMDWF & MMWWF	Checked by		Date	

Background Data for Developing DEQ Graph #1

- use two closest weather stations & Avg. Values (Dallas & McNary Field)

$$MMDWF_{10} = \frac{8.40 + 11.44 \text{ inch/day}}{2} = 9.92 \text{ inch/day}$$

$$MMDWF_{5} = \frac{4.03 + 4.47}{2} = 4.25 \text{ inch/day}$$

- Develop Graph using 2011-2014 DMR Data

$MMWWF_5 = 2.21$ mgd

$MMDWF_{10} = 1.42$ mgd

U.S. Department of Commerce
National Oceanic & Atmospheric Administration
National Environmental Satellite, Data,
and Information Service

Climatography of the United States No. 20 1971-2000

National Climatic Data Center
Federal Building
151 Patton Avenue
Asheville, North Carolina 28801
www.ncdc.noaa.gov

Station: SALEM MCNARY AP, OR

COOP ID: 357500

Climate Division: OR 2 **NWS Call Sign: SLE**

Elevation: 205 Feet Lat: 44°54N Lon: 123°00W

5-YR JAN

Precipitation (inches)

	Precipitation Totals									Mean Number of Days (3)				Precipitation Probabilities (1) Probability that the monthly/annual precipitation will be equal to or less than the indicated amount										
	Means/ Medians(1)		Extremes							Daily Precipitation				Monthly/Annual Precipitation vs Probability Levels These values were determined from the incomplete gamma distribution										
Month	Mean	Med-ian	Highest Daily(2)	Year	Day	Highest Monthly(1)	Year	Lowest Monthly(1)	Year	>= 0.01	>= 0.10	>= 0.50	>= 1.00	.05	.10	.20	.30	.40	.50	.60	.70	.80	.90	.95
Jan	5.84	6.06	3.30	1933	2	10.89	1974	.41	1985	17.3	12.0	3.8	1.2	1.49	2.04	2.88	3.63	4.37	5.15	6.02	7.05	8.40	10.52	12.51
Feb	5.09	4.41	2.99	1937	1	13.01	1996	.75	1988	15.8	11.5	3.4	.8	1.40	1.88	2.62	3.26	3.89	4.54	5.27	6.14	7.26	9.02	10.66
Mar	4.17	3.73	2.55	1943	31	8.56	1983	.89	1992	16.9	11.3	2.4	.3	1.53	1.92	2.49	2.96	3.40	3.86	4.36	4.94	5.68	6.82	7.86
Apr	2.76	2.54	2.21	1937	13	5.72	1996	.62	1977	14.0	8.3	1.3	.1	.84	1.11	1.50	1.83	2.16	2.50	2.87	3.30	3.87	4.75	5.57
May	2.13	1.92	1.76	1991	17	5.56	1998	.05	1992	11.3	6.2	1.0	.1	.44	.63	.94	1.23	1.52	1.82	2.17	2.59	3.14	4.03	4.86
Jun	1.45	1.13	1.63	1985	6	4.19	1984	.26	1987	7.7	3.9	.7	.2	.31	.44	.66	.85	1.04	1.25	1.48	1.76	2.13	2.72	3.27
Jul	.57	.33	1.80	1987	18	2.63	1983	.00+	1994	3.4	1.6	.2	@	.00	.01	.06	.13	.22	.34	.48	.67	.95	1.43	1.93
Aug	.68	.41	1.14	1971	31	2.56	1978	.00+	1988	3.9	2.0	.2	@	.00	.00	.04	.13	.24	.38	.57	.81	1.16	1.77	2.39
Sep	1.43	1.16	1.76	1969	17	3.98	1971	.00	1975	6.7	3.8	.8	.2	.02	.10	.27	.46	.69	.96	1.30	1.73	2.34	3.39	4.45
Oct	3.03	2.62	2.71	1994	31	7.96	1994	.12	1988	11.0	6.8	1.9	.4	.43	.68	1.12	1.54	1.98	2.46	3.02	3.70	4.62	6.12	7.56
Nov	6.39	6.00	2.85	1996	19	15.23	1973	1.13	1976	18.2	12.9	4.3	1.3	1.83	2.44	3.36	4.15	4.92	5.73	6.63	7.68	9.04	11.18	13.17
Dec	6.46	6.67	4.30	1933	6	15.01	1996	1.26	1976	18.0	12.5	4.7	1.5	1.98	2.59	3.51	4.30	5.06	5.85	6.72	7.75	9.07	11.13	13.04
Ann	40.00	39.07	4.30	Dec 1933	6	15.23	Nov 1973	.00+	Jul 1994	144.2	92.8	24.7	6.1	26.08	28.68	32.07	34.67	37.01	39.28	41.65	44.29	47.51	52.23	56.35

· Also occurred on an earlier date(s)
Denotes amounts of a trace
) Denotes mean number of days greater than 0 but less than .05
* Statistics not computed because less than six years out of thirty had measurable precipitation

(1) From the 1971-2000 Monthly Normals
(2) Derived from station's available digital record: 1928-2001
(3) Derived from 1971-2000 serially complete daily data
Complete documentation available from:
www.ncdc.noaa.gov/oa/climate/normals/usnormals.html

10-YR MAY

U.S. Department of Commerce
National Oceanic & Atmospheric Administration
National Environmental Satellite, Data,
and Information Service

Climatography of the United States No. 20 1971-2000

National Climatic Data Center
Federal Building
151 Patton Avenue
Asheville, North Carolina 28801
www.ncdc.noaa.gov

Station: DALLAS 2 NE, OR

COOP ID: 352112

Climate Division: OR 2 **NWS Call Sign:**

Elevation: 290 Feet Lat: 44°57N Lon: 123°18W

5-YR, JAN

Precipitation (inches)

Month	Precipitation Totals: Means/Medians(1): Mean	Median	Extremes: Highest Daily(2)	Year	Day	Highest Monthly(1)	Year	Lowest Monthly(1)	Year	Mean Number of Days (3) Daily Precipitation: >= 0.01	>= 0.10	>= 0.50	>= 1.00	Precipitation Probabilities (1) Monthly/Annual Precipitation vs Probability Levels (These values were determined from the incomplete gamma distribution): .05	.10	.20	.30	.40	.50	.60	.70	.80	.90	.95
Jan	7.82	8.52	3.91	1974	15	13.71	1995	.26	1985	17.7	12.2	5.2	1.8	1.74	2.46	3.61	4.64	5.68	6.78	8.02	9.50	11.44	14.54	17.47
Feb	6.66	6.28	3.23	1996	6	17.05	1999	1.29	1988	15.5	10.6	4.3	1.3	1.80	2.43	3.39	4.23	5.06	5.93	6.89	8.03	9.52	11.85	14.03
Mar	5.33	5.14	2.30	1943	31	11.18	1983	1.10	1992	16.4	11.1	3.1	.7	1.92	2.42	3.14	3.75	4.33	4.92	5.57	6.32	7.28	8.77	10.13
Apr	3.24	2.63	2.33	1937	13	7.20	1996	.67	1977	12.8	7.4	1.8	.3	.84	1.14	1.61	2.03	2.44	2.87	3.35	3.91	4.66	5.82	6.92
May	2.21	2.05	1.57	1991	17	5.80	1998	.02	1992	10.3	5.7	1.0	.2	.31	.49	.81	1.12	1.44	1.79	2.20	2.70	3.37	4.47	5.53
Jun	1.41	1.19	1.53	2001	27	3.78	1981	.10	1987	7.2	3.9	.7	.1	.28	.41	.62	.81	1.00	1.21	1.44	1.72	2.09	2.69	3.25
Jul	.50	.32	1.75	1987	18	2.05	1987	.00+	1994	3.3	1.3	.2	.1	.00	.00	.06	.14	.22	.33	.45	.61	.84	1.22	1.61
Aug	.67	.55	1.21	1954	30	1.73+	1978	.00+	1998	3.5	1.8	.4	.0	.00	.00	.06	.15	.26	.40	.57	.80	1.13	1.70	2.28
Sep	1.44	.96	1.42	1969	18	4.01	1978	.00	1975	6.5	3.7	1.0	@	.03	.13	.31	.52	.75	1.02	1.34	1.75	2.33	3.31	4.28
Oct	3.28	2.98	3.92	1994	27	7.43	1975	.13	1988	10.4	6.1	2.3	.6	.47	.74	1.21	1.67	2.14	2.66	3.27	4.01	5.01	6.64	8.20
Nov	7.79	7.23	4.20	1996	19	21.24	1973	1.55	1976	17.5	13.0	4.8	1.5	2.09	2.83	3.95	4.94	5.91	6.93	8.06	9.40	11.15	13.88	16.44
Dec	8.78	8.65	4.32	1964	22	21.93	1996	1.83	1976	17.6	12.7	6.0	2.4	2.68	3.51	4.76	5.83	6.87	7.94	9.13	10.52	12.32	15.13	17.73
Ann	49.13	48.41	4.32	Dec 1964	22	21.93	Dec 1996	.00+	Aug 1998	138.7	89.5	30.8	9.0	31.28	34.59	38.91	42.25	45.25	48.18	51.23	54.64	58.81	64.93	70.29

- Also occurred on an earlier date(s)
! Denotes amounts of a trace
@ Denotes mean number of days greater than 0 but less than .05
* Statistics not computed because less than six years out of thirty had measurable precipitation

(1) From the 1971-2000 Monthly Normals
(2) Derived from station's available digital record: 1928-2001
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Complete documentation available from:
www.ncdc.noaa.gov/oa/climate/normals/usnormals.html

10-YR MAY

APPENDIX C

Water Quality Trading Memorandum
September 14, 2011

WINZLER & KELLY
15575 SW Sequoia Pkwy, Ste. 140
Portland, OR 97224-7233

September 14, 2011

MEMORANDUM

Project No.: 12149-10001

Project Name: Sanitary Sewer System Facilities Plan – Water Quality Trading

To: Mike Danko

From: Don Whitehead, P.E.

Copies To:

Subject: Application of Water Quality Trading To Projected Future Growth

BACKGROUND

As a follow-up task to the Draft Facilities Plan (July 28, 2009) Winzler & Kelly was asked to review the potential for pollution credit trading as part of a strategy to address projected future growth.

A meeting was held at the City of Independence City Hall on May 19, 2011 with representatives of Oregon Department of Environmental Quality (DEQ), City staff and Winzler & Kelly, to discuss the potential application of Water Quality Trading. Specific concerns related to the segment of the Willamette River where the City's NPDES permitted discharge is located were noted. At the meeting some of the larger permitted industrial discharges upstream from the City were identified. DEQ indicated that they would be open to a proposal for Water Quality Trading, if the City is able to develop a partnership and clearly identify the pollution parameters that will be involved.

STATUS OF EXISTING SEWERAGE SYSTEM

Remaining Capacity

There are two critical capacity concerns related to the treatment facility:

1. Biological treatment
2. Summer storage capacity

CITY NPDES PERMIT

Permit Requirements

DEQ permits the City to discharge to the Willamette River under requirements, standards and conditions outlined in NPDES Permit No. 990232. All conditions of the permit must be met. The permit is currently written to address the treatment efficiencies and characteristics inherent to the City's facultative wastewater treatment lagoons. The permitted outfall is shared with the City of Monmouth and discharge to the Willamette River is limited to the months of November

through May. The permit requirements have been developed with consideration for the technical limits of the treatment facility and the water quality of the receiving stream. A number of additional water quality parameters are under consideration at this time and continued close coordination and communication with DEQ representatives will be critical to developing an integrated treatment approach that will meet current and future permit requirements.

The approach outlined in this memorandum represents the current understanding of the required treatment and applicable water quality parameters. If the process of negotiating Water Quality Trading proceeds, input from DEQ will be important to confirm that all requirements are met. At this time the following regulated limitations are identified in the City's NPDES permit:

Permit Limitations

- 5-Day Biochemical Oxygen Demand (BOD_5)
- Total Suspended Solids (TSS)
- E. coli Bacteria
- pH
- BOD_5 Removal Efficiency
- TSS Removal Efficiency
- Total Chlorine Residual

In addition the permit prohibits discharge to the receiving stream during the months of June through October. Wet weather mass load limits are based on a design flow of 2.0 mgd. A new permit limit on Excess Thermal Load will be applied from October 15 through May 15 each year.

Additional Monitoring Requirements and Potential Additional Permit Limitations

In addition to the permit limitations noted above, the City must monitor for the following:

- Iron annually in January.
- Mercury annually in January.
- Ammonia and dissolved oxygen monthly during the discharge season.
- Excess Thermal Load calculated based on weekly average effluent temperature during the time frame of November through May.

DEQ PROGRAM FOR WATER QUALITY TRADING UNDER NPDES PERMITS

DEQ has developed an Internal Management Directive (IMD) for water quality trading (*Water Quality Trading in NPDES Permits, Internal Management Directive, December – 2009*). The IMD is intended to

"define concepts, explain eligibility, and describe specific trading scenarios that DEQ anticipates and generally supports."

One of the Basic Requirements for the program (Section 2, page 7) is that water quality trading can only be applied to meet existing federal treatment technology requirements, including EPA secondary treatment standards for publicly-owned treatment works (POTW's). The definition of "secondary treatment standards" is provided by EPA and includes the possibility of special allowances and standards for permits of stabilization lagoons. However, EPA guidelines include clear language that special consideration and/or modified standards cannot be provided if a

treatment facility is operating beyond its hydraulic or organic loading limit.

"A treatment works that is operating beyond its design hydraulic or organic loading limit is not considered an eligible facility. If overloading or structural failure is causing poor performance, the solution to the problem is construction, not effluent limitations adjustment." (EPA NPDES Permit Writers Manual)

Water Quality Trading Opportunities for the City of Independence

The wastewater treatment facility must be operated at a design loading consistent with normal standard of care and traditional design parameters. We do not believe that the City will be allowed to overload the existing lagoons either hydraulically or organically, and trade for equivalent mass loads associated with increasingly poor treatment performance. Furthermore, we believe that intentional overloading of the lagoons will produce undesirable and unpredictable side effects including:

- Offensive odors.
- High solids accumulation in the primary lagoon cells.
- Mass load increases in effluent discharge to the Willamette River that will be difficult to predict and could rapidly, perhaps exponentially, increase to approach those associated with only primary treatment/settlement.

With these limitations noted we believe the opportunity for water quality trading is likely to be in the following two areas:

1. Anticipated mass load increases to the Willamette River associated with population growth during winter months.
2. Dry weather (May 1 – October 31) discharges to the Willamette River. These are not currently allowed and this change would provide an alternative to increasing lagoon storage volume, or treated effluent reuse (spray irrigation).

Potential Trading Partnerships

Since the May 19th meeting, W&K has been in contact with several industrial permit holders to inquire about the possibility of water quality trading (WQT). With the assistance of DEQ staff we identified on a focused group of industrial users who have either closed or modified their operations in a manner that has reduced their water pollutant loads to the Willamette River.

The entities contacted included the following:

- International Paper/Weyerhauser – Albany/Millersburg Mill
- Georgia Pacific – Halsey Paper Mill
- International Paper – Springfield Mill
- Evanite Fiber Corporation – Corvallis

International Paper and Georgia Pacific indicated that they were not interested in trading or selling all or a portion of their permitted discharge loads to the receiving stream. Evanite Fiber Corporation holds a permitted year round discharge but due to changes in their operations, they no longer discharge a detectable BOD_5 load. The Director of Fiber Operations in Corvallis indicated that they would consider an offer to sell some or all of this and requested that the City outline a written proposal.

Proposed Water Quality Trading Requests – BOD_5

Attached to this memorandum is a matrix summarizing the Permitted and Requested BOD_5 loads associated with the proposed Water Quality Trading outlined in the following paragraphs. Also included in the table is a summary of the available BOD_5 loads that Evanite Fiber is allowed under their current NPDES permit. The approach to developing these numbers is detailed in the following paragraphs:

Winter Month Mass Load Increases Due to Population Growth

The current permitted load for BOD_5 is based on a design flow of 2.0 mgd at an assumed treated effluent concentration of 30 mg/l (monthly average) and 45 mg/l (weekly average) and produces the following limits:

Permitted Loads

BOD_5	500 lbs/day	Monthly Average
	750 lbs/day	Weekly Average
	1000 lbs	Daily Maximum

Although the estimated future average day wet weather flows are projected to be 1.83 mgd which will remain below the design flow of 2.0 mgd, it seems prudent to include an increase in winter BOD_5 allowances as part of a WQT agreement. The ratio of projected to current population (2030/2011) is 1.64. Applying this to the current permitted loads produces the following:

Requested Permitted Loads

BOD_5	820 lbs/day	Monthly Average
	1230 lbs/day	Weekly Average
	1640 lbs	Daily Maximum

The net result would be a requested trade or purchase of:

Proposed Water Quality Trade

BOD_5	320 lbs/day	Monthly Average
	480 lbs/day	Weekly Average
	640 lbs	Daily Maximum

Dry Weather Mass Load

Since the current permit does not allow discharge of treated effluent during the dry weather months of June through October, any discharge will be a requested increase in pollutant load(s) and correspondingly an opportunity to apply a water quality trade. The most conservative approach would be to request a mass load equivalent to the total projected dry weather flow in the year 2030. This is conservative in that in theory it would not require that the City store any flow during the dry weather months. This would allow for the potential decommissioning of some of the existing lagoon area in conjunction with development of a higher level of treatment for future growth. Projected average day dry weather flows (ADWF) are 1.02 mgd in the year 2030. An assumed treated effluent concentration of 30 mg/l (monthly average) and 45 mg/l (weekly average) produces the following projected loads:

Requested Permitted Loads and Proposed Water Quality Trade

BOD_5	255 lbs/day	Monthly Average
	383 lbs/day	Weekly Average
	510 lbs	Daily Maximum

Evanite Fiber holds a NPDES permit that allows for BOD_5 to be discharged at the following maximum loads:

Evanite Fiber Permitted Loads – (Wet Weather, November – May)

BOD_5	1400 lbs/day	Monthly Average
	2100 lbs	Daily Maximum

Evanite Fiber Permitted Loads – (Dry Weather, June – October)

BOD_5	850 lbs/day	Monthly Average
	1300 lbs	Daily Maximum

During discussions with representatives of Evanite Fiber in June and July of 2011 they indicated a willingness to consider a WQT. It appears that the requested WQT outlined in the previous paragraphs would be well within the BOD_5 limits of their permit.

OTHER WATER QUALITY PARAMETERS

A potential WQT for BOD_5 addresses one important parameter. However, by itself it is not likely to be sufficient to convince DEQ to allow the City to begin discharging during dry weather months. A likely scenario is that the City will need to propose a comprehensive treatment approach that includes a higher level of treatment such as can be produced by mechanical plants (Activated Sludge, Sequencing Batch Reactors, Membrane Bioreactors) to address traditional pollutant parameters such as TSS and "pollutants of emerging concern".

An additional area of potential concern noted during permit discussions with DEQ is the impact of thermal pollution during the October 15 through April 15 time period associated with fish spawning and rearing. The current permit for the City of Independence only considers thermal impacts during the wet weather discharge. During the wet weather months DEQ has estimated that there is a "low potential" for permitted effluent discharge to increase the receiving stream temperature more than the 0.5 degrees Celsius limit.

Dry weather discharge will likely involve a different standard and will require hydraulic/thermal modeling and possibly a different approach to effluent discharge. One possible discharge option to address dry weather thermal impacts is a hyporheic discharge which involves discharge to a gravel stratum adjacent to and along a river where surface ground water mixes with river water. As noted in the Draft Facilities Plan such an approach will raise additional requirements for permitting, ground water monitoring and potentially a higher level of treatment.

To date DEQ has indicated a dry weather discharge to the Willamette River was possible but highly unlikely regardless of the treatment approach. However, a water quality trade for BOD_5 could be the cornerstone of an integrated approach that will result in a cost effective approach with the least capital cost to the City. If properly structured and documented it could provide DEQ with assurance that summer discharge by the City will not reduce the overall water quality.

Attachment: ***Table 1 – Potential Water Quality Trading – Biochemical Oxygen Demand (BOD_5)***

TABLE 1 - POTENTIAL WATER QUALITY TRADING - Biochemical Oxygen Demand (BOD_5)

CITY OF INDEPENDENCE, OREGON
14-Sep-11

BOD - WET WEATHER MONTHS (November - May)	**City of Independence**				**Evanite Fiber**		
	Permitted Loads	**Requested Permitted Loads**	***Proposed Water Quality Trade***		**Permitted Loads**	**Proposed Water Quality Trade**	***Percentage of Permitted Load***
Monthly Average (lbs/day)	500	820	***320***		1400	320	***23%***
Weekly Average (lbs/day)	750	1230	***480***		0		
Daily Maximum (lbs)	1000	1640	***640***		2100	640	***30%***
BOD - DRY WEATHER MONTHS (June - October)	**Permitted Loads**	**Requested Permitted Loads**	***Proposed Water Quality Trade***		**Permitted Loads**	**Proposed Water Quality Trade**	***Percentage of Permitted Load***
Monthly Average (lbs/day)	0	255	***255***		850	255	***30%***
Weekly Average (lbs/day)	0	383	***383***		0		
Daily Maximum (lbs)	0	510	***510***		1300	510	***39%***

APPENDIX D

Lagoon Storage Summary and Water Balance

INDEPENDENCE, OREGON
DRY WEATHER WASTEWATER TREATMENT FACILITY
STORAGE VOLUME RECORD SUMMARY JUNE - OCTOBER (2008-2014)
12/23/2014

LAGOON WATER BALANCE
USING MEASURED INFLUENT AND PRECIPITATION & ESTIMATED INFILTRATION

Year	Total Influent (1) MG	Total Effluent (2) MG	Precipitation MG	Infiltration MG	Evaporation MG	Change In Lagoon Volume(3) MG	Lagoon Storage Capacity Volume (4) MG	Reqd Additional Volume to Dispose MG
2008	84.3	0.0	4.6	-53.0	-35.7	**0.3**	52.0	
2009	84.6	-10.7	8.2	-53.0	-34.4	**-5.2**	52.0	
2010	97.9	-65.8	13.9	-53.0	-31.5	**-38.5**	52.0	
2011	147.0	-48.6	6.1	-53.0	-34.3	**17.3**	52.0	**13.9**
2012	153.8	-13.5	12.3	-53.0	-37.2	**62.4**	52.0	**23.9**
2013	143.5	-53.4	12.1	-53.0	-31.7	**17.5**	52.0	**18.9**
2014	141.5	-33.7	6.9	-53.0	-35.5	**26.3**	52.0	**8.0**
								16.2

Evap (6) (Inch)	Precip (5) (Inches)	Time Period
34.6	3.33	**June 1 - October 31**
33.34	5.94	**June 1 - October 31**
30.56	10.02	**June 1 - October 31**
33.3	4.44	**June 1 - October 31**
36.11	8.87	**June 1 - October 31**
30.8	8.75	**June 1 - October 31**
34.4	5.00	**June 1 - October 31**

Rate of Infiltration = *0.25 inches/day 0.346 mg/day*
Evaporation Pan Coeff. = *0.745*

Average Dry Weather Precip. Total =7.0 inches

(1) *Total Influent recoreded and reported on DMR (June 1 - October 30).*
(2) *Total Effluent recorded and reported on DMR (Early Discharge prior to October 31).*
(3) *Total Water Balance based on measured influent,effluent, precipitation, evaporation and seepage (June 1 - October 31).*
(4) *Total Residual Storage Capacity of Lagoons accounting for measured sludge accumulation.*
(5) *Total Precipitation recorded between June 1 - October 31 at lagoons.*
(6) *Total Evaporation recorded June 1 - October 31 at Corvallis weather station.*

APPENDIX E

Meeting Minutes – November 26, 2008
Independence/Monmouth Planning Workshop

INDP0000-0031
MA: 3
Meeting Date: 11-13-08
Meeting Time: 9:00 am – 4:00 pm
Meeting Location: City of Independence Library

MEETING MINUTES – Revised 11-26-08

These minutes were prepared by David Evans & Associates, Inc. for the wastewater facilities planning workshop for the Cities of Independence and Monmouth, Oregon. Edits were provided by Whitaker Engineering and Clearwater Engineering and incorporated into this revised set of Minutes.

Project: **Sanitary Sewerage System Facilities Plan Update**
Engineer: **David Evans and Associates, Inc.**
Whitaker Engineering, Inc.

Attendees:
Ken Perkins, Kie Cottam, Chad Tracy, AaronWimer – City of Independence, Oregon
Craig Johns, Russ Cooper – City of Monmouth, Oregon
Don Whitehead - David Evans and Associates, Inc.
Todd Whitaker – Whitaker Engineering, Inc.
Dave Crimp - Clearwater Engineering
Steve Celeste – Kennedy Jenks Consulting
Ray Bartlett – Economic and Financial Analysis (12:00 Noon to 1:30 pm)

1. INTRODUCTIONS

- Purpose for Meeting and Expected Outcome

  This is the first opportunity for the two communities to begin a formal discussion identifying potential opportunities and problems, and framing the financial impacts associated with a regional facility. At the completion of the meeting the group will offer their recommendation whether or not a regional facility warrants further consideration and investigation. The group will also develop recommended action items.

- Suggested ground rules were identified to encourage open discussion.

2. COMMUNICATION

Key contacts for the two cities were identified as:

- City of Independence - Mike Danko, Don Whitehead (David Evans and Associates)
- City of Monmouth – Craig Johns, Todd Whitaker (Whitaker Engineering)

3. EVALUATION CRITERIA

Prior to beginning thediscussion of the technical requirements for a regional facility, the group discussed a method for evaluating the merits of treatment approaches. DEA provided an example of a decision matrix and sample evaluation criteria that could be used to rank and weight individual criteria. The larger group broke into two smaller groups to brainstorm additional criteria that may be beneficial for comparing multiple alternatives. After summarizing the combined criteria each member of the group identified their top five with the results tallied on the attached Draft Decision Matrix.

Attachment A: Draft Decision Matrix – Wastewater Treatment Alternatives

4. POPULATION FLOWS/LOADS – Current and Projected

Whitaker Engineering provided a preliminary summary of estimated Design Flows for 2030. These projections were noted to be preliminary and valuable to initiate discussions and comparison of alternative treatment processes. These flow estimates will be reviewed and refined by DEA and Whitaker Engineering for Independence and Monmouth respectively in the coming weeks.

Attachment B: Summary of Design Flows

5. SUMMARY OF REGULATORY CONSIDERATIONS

Kennedy Jenks provided a handout summarizing regulatory issues and permit elements that may be part of future NPDES permit discussions with Oregon DEQ. Each of the items was discussed briefly with acknowledgement that regardless of whether or not the Cities develop a combined treatment facility, DEQ permitting staff should be part of the discussion and evaluation. It was noted that although summer discharge to the Willamette River is not prohibited, there are a number of current and anticipated regulatory requirements that will make this difficult to accomplish regardless of the treatment technology selected.

Attachment C: Monmouth/Independence Regionalization Regulatory Summary

6. REGIONAL TREATMENT ALTERNATIVES DEVELOPMENT

DEA provided Schematic Diagrams describing two alternate configurations that could be developed as regional treatment facilities. The two diagrams identified centralized and satellite configurations described in Alternative A and B respectively.

Whitaker Engineering and Kennedy Jenks provided example process diagrams for Advanced Lagoons, Activated Sludge (AS), Membrane Bioreactor (MBR), Oxidation Ditch (OXD), and Sequencing Batch Reactor (SBR) treatment processes.

A brief description of the major unit processes involved with each treatment approach was provided.

Attachment D: Alternative Regional Treatment Configuration Diagrams A and B
Attachment E: Process Diagrams for wastewater treatment AS, MBF, OXD and SBR

7. COST ESTIMATING:

Whitaker Engineering developed budget level cost estimates in anticipation of this meeting and provided a written summary. The cost estimates were developed from reported construction costs for wastewater treatment facilities using AS, MBR, OXD and SBR treatment technologies. It was noted that these are very preliminary and conservative construction cost estimates and more detailed cost analysis and estimating will be required to address the specific requirements and opportunities of the two cities.

Single Phase Construction – Abandon Existing Wastewater Treatment Facilities
These preliminary estimates for Single Phase Construction assume that both communities would abandon the existing WWTF's and build a new facility(s). Complete abandonment of the existing treatment facilities represents significantly higher capital cost than a phased approach that continues to utilize all or part of the existing treatment capacity. For this reason there will be strong motivation to implement a phased approach that continues to utilize all or part of the existing treatment capacity. Although single phase regionalization may not be possible due to the higher initial cost, is does provide a common basis to begin comparing alternate approaches (regional –vs- separate) and alternate treatment technologies.

The first three alternatives considered assume improvements will be constructed in a single phase. Using this cost estimating approach, Full Regionalization at $80m appears to be the alternative with the least cost (initial capital cost) when compared to Partial Regionalization and Individual Plant development which had estimated totals of $85m and $95m respectively.

Multiple Phase Construction – Utilize Existing Wastewater Treatment Facilities
An additional Option (Option 4) was presented which identified development of a regional plant occurring in two phases. During Phase I the existing lagoons at each City would be converted to aerated lagoons and a regional effluent reuse facility would be utilized by both Cities to accommodate summer discharges. During Phase II a new centralized regional plant including a solids management facility would be constructed. The totals for Phases I and II are estimated to be $49m and $54m respectively ($103m total).

Attachment F: Power Point outline summary of cost estimate
Attachment G: Historic cost estimate table and graph

8. FINANCING

Ray Bartlett with Economic and Financial Analysis joined the meeting at 12:00 noon and provided a summary of organizational and financial considerations under a regional approach. Potential administrative formats include:

- ***Intergovernmental Agreement***: This could be developed either with City identified as the single owner and the second city as a paying user, or a structure where both cities are mutual partners. Ray cautioned that there can be some potential for future disagreement over compensation for unused capacity if the two communities grow at different rates.

- ***"190" District***: where both cities adopt ordinances that complement and support a sanitary sewer district. Under this approach the district can issue revenue bonds.

- ***"405" District***: which is set up as a service district with its own board of directors and no direct control by the individual cities. The district can issue either revenue or general obligation bonds. Examples of existing 405 Districts noted were the MWMC in Eugene/Springfield and Cleanwater Services in Washington County.

Additional observations include:

- Of the three structures Ray thought the 190 District may be best suited for the two cities.
- It should be agreed who owns and maintains the collection and treatment facilities.
- Phasing would be easiest if both cities adopt the same ordinances.

Major funding sources for this type of project include:

- State Revolving Loan Fund (SRF): Loans are limited to municipalities and districts with highest need.
- Rural Development (RD): Is currently the only agency with an approved federal budget.
- Oregon Economic Development Department (OEDD): State funding.
- General Obligation Bonds (GO): Require a vote.
- Revenue Bonds: Assessed to the users of the system are tied to cost, and do not require a vote.
- Systems Development Charges (SDC): Can be used to pay for capital improvements if the methodology is well defined for that use.

All funding options are sensitive to the current economic climate and subject to changes in the coming weeks and months. The first three sources noted (SRF, RD and OEDD) as currently limited.

From a funding perspective it may be desirable to construct in phases which would allow for multiple RD loans that can be retired separately.

For comparison with other communities Ray noted that user fees ranging from $40 to $80/month are common with some as high as $120/month. Independence is currently in the range of $24/month.

Another source of grants or loans discussed was energy production tax credits which would likely require a large scale development of alternative energy production or co-generation.

9. REQUIRED STEPS FOR FACILITIES PLANNING THROUGH CONSTRUCTION

- Facilities Planning document preparation with a focus on comparing treatment alternatives and utilizing a present worth analysis are currently underway for both cities.
- Environmental planning and analysis will be required for any project that includes federal funding or any federally regulated permit. The environmental planning process will incorporate additional evaluation criteria beyond a present worth analysis.
- Including the public in the evaluation process is important, and will need to be planned, accounted for, and documented.

10. PRELIMINARY CONCLUSIONS

- Financing will be a challenge regardless of the approach selected.
- If a regional approach is pursued, a phased regional development appears to be more achievable have greater flexibility.
- The existing facilities should be used to the maximum extent possible.
- I/I reduction may provide some additional hydraulic capacity (recovery of capacity) but additional data and investigation is required to confirm the magnitude and cost effectiveness.
- Money will be tight and phasing options need to be more fully developed and considered.
- Short term/initial options should be developed.
- For the City of Monmouth, treatment capacity is a critical need.
- For the City of Independence, summer storage capacity is the critical need.
- "Sharpen" the pencil for the cost estimates associated with the long term plan.
- A timeline and plan for communicating with City Managers, City Councils and the General Public should be developed.
- A summary of operation and maintenance (O&M) costs should be developed for long term alternatives to allow for a more complete cost comparison.
- A memorandum from Ray Bartlett providing more detail regarding the regional strategies for structure and financing would be valuable to assist with evaluation of future alternatives.
- A phased approach will save <u>initial</u> capital cost, allow cities more time to develop funds and allow the application of treatment capacities and technologies that are most suitable to the nature of development.
- A disadvantage to the phased approach is that it likely will result in the highest <u>total</u> capital cost.

11. FOLLOW-UP ACTION AND FEEDBACK

- A check-in with City Managers should be scheduled to include: Mike Danko, Craig Johns, Don Whitehead and Todd Whitaker.
- Refined alternatives and additional details should be developed for phased development of a regional facility.
- A preliminary schedule should be developed for a phased development of a regional treatment plant.
- A plan and schedule should be developed to communicate and solicit feed-back from the City Councils.
- A plan and schedule should be developed to communicate and solicit feedback from the public.
- The impact on user fees should be calculated for specific alternatives and phases.

Attachments are noted in the previous paragraphs and included as part of these minutes.

DRAFT Decision Matrix - Wastewater Treatment Alternatives
Developed at November 13, 2008 Workshop

Evaluation Criteria	Weighting of Importance	BENEFIT IN IMPLEMENTING GOAL			Votes for Importance at 11/13/08 Meeting
		Alternate No. 1	Alternate No. 2	Alternate No. 3	
		Lagoon	Membrane Treatment (MR)	Sequencing Batch Reactor (SBR)	
1 Ease of Accommodating Phased Growth					****
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
2 Ease of Permitting					
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
3 Effectiveness at Achieving Effluent Water Quality					******
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
4 Reliability					*****
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
5 Initial Capital Cost to Build (Affordable)					******
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
6 Improve Site Aesthetics					
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
7 Environmental Impacts					
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
8 Conserve Potable Water					
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
9 Conserve/Generate Energy					
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
10 Annual Cost to Operate (Rates)					*******
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
11 Integrate with Open Space					*
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
12 Administrative Efficiency for Combined Facility					*
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	

ATTACHMENT A

Evaluation Criteria	Weighting of Importance	BENEFIT IN IMPLEMENTING GOAL			Votes for Importance at 11/13/08 Meeting
		Alternate No. 1	Alternate No. 2	Alternate No. 3	
		Lagoon	Membrane Treatment (MR)	Sequencing Batch Reactor (SBR)	
13 Solve Immediate Capacity Problems					
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
14 Land Requirements/ Opportunities					
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
15 Accessibility to Additional Funding Sources					****
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
16 Adaptability to Future Regulatory Changes					***
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
17 Community Support					***
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
18 Utilization of Existing Staff/Certifications					**
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
19 Ease of Transition					
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
20 Maximize Resource Potential					**
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
21 Timing					**
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
22 Site/Location Considerations					
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	
23 Operator/Public Safety					*
Raw Score		0	0	0	
Weighted Score	100%	0	0	0	

Legend
2 = Much Better Than Baseline
1 = Better Than Baseline
0 = Same As Baseline (Existing Lagoon Treatment/Agricultural Spray Irrigation)
-1 = Worse Than Baseline
-2 = Much Worse Than Baseline

ATTACHMENT B

PRELIMINARY

2030 Design Flows

Item	Description	Independence	Monmouth	Combined	Comments
ADWF, mgd	Average Dry Weather Flow	1.2	1.4	2.6	Not used for sizing facilities
AAF, mgd	Average Annual Flow	1.8	2.1	3.9	Not used for sizing facilities
MDDWF, mgd	Max Daily Dry Weather Flow	1.8	2.1	3.9	Tertiary treatment facilities (filters)
$MMWWF_{10}$, mgd	Max Monthly Avg Daily Flow, 10-Yr Event	3	3.6	6.6	Biological Treatment, Solids Processing
PDF_5, mgd	Peak Daily Flow, 5-Yr Event	6.1	7.5	13.6	Headworks, all process basins
PIF_5, mgd	Peak Hourly Flow, 5-yr Event	12.3	15.2	27.5	Influent Pump Station(s)

ATTACHMENT C

Monmouth/Independence Regionalization
Regulatory Summary
November 13, 2008

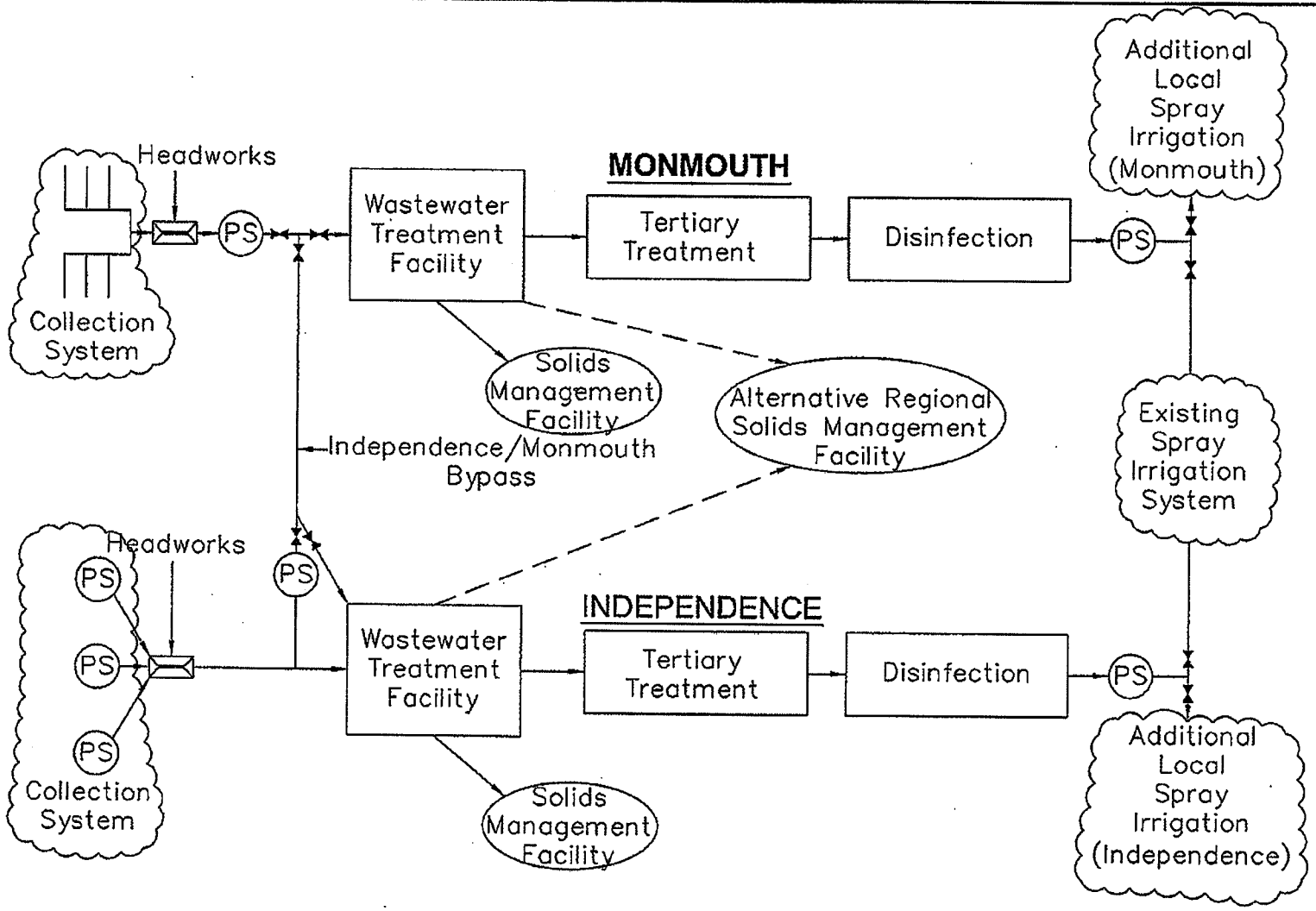
Item	Description	Issues
NPDES permit	Permit has expired and renewal application has been submitted	Dry weather discharges not allowed. Construction of mechanical plant would likely decrease mass discharge limits. Chlorine limits likely to be more restrictive, requiring dechlorination.
Willamette River TMDL	TMDL addresses temperature, bacteria, and mercury	Thermal limits—potential thermal limits added to permit. Bacteria—minimal impact to discharges. Mercury—monitoring may be required.
Temperature	TMDL allocates thermal load for "small point sources".	Small point sources allocated 95M kcal for summer. Small point sources allocated 216M kcal for spawning season (10/15 – 5/15). Thermal WLA not yet divided up between various small point sources.
303 (d) listings and potential future TMDLs	303 (d) listings: DO Iron Phosphorus (summer) Alkalinity Antimony Barium Cadmium Chloride Chlorophyll-a Lead Selenium Silver Ammonia pH	More stringent treatment requirements. Possible metals limits could eventually trigger need for RO. Possible nutrient limits could trigger need for nutrient removal facilities. TMDL updates (parameters added) expected every 5 years.
Willamette Basin standards (OAR 340-041-0345)	Base standards for treatment and water quality	Requires 10 mg/L effluent TSS/BOD during dry weather, 30 mg/L wet weather for mechanical treatment plant.
Toxicity and Reasonable Potential Analysis	Numeric criteria for 100 toxic substances	RPA required to assess toxics. Action required if reasonable potential for toxicity.

(RPA) IMD		Toxics monitoring requirements increase greatly if design ADWF is >1 mgd—lab costs estimated at over $50,000. Dechlorination likely will be required. More stringent toxics standards are being considered by EPA. DEQ is currently evaluating toxics in the Willamette River, which could trigger additional monitoring.
Mixing zone IMD	New regulations for evaluating mixing zones	Increases complexity of mixing zone evaluations. Requires assessment of numerous toxic substances. Mixing zone will likely decrease in size, reducing available dilution. Could trigger effluent ammonia limits. Could trigger limits for other toxics.
Ammonia	Ammonia toxicity calculation methodology likely to change.	Potential effluent ammonia limits. Lagoons provide limited ammonia removal.
Mixing zones	Periodic legislative attempts to eliminate mixing zones	Elimination of mixing zones would require compliance with all water quality criteria at end of pipe.
Contaminants of emerging concern (CEC)	Trace contaminants causing water quality problems	CEC receiving increased publicity and generating more public concern. Impacts to aquatic life have been documented. Examples: caffeine, steroids, hormones, antibiotics, prescription drugs, illicit drugs.
Persistent bioaccumulative toxins (PBTs)	Certain toxic substances degrade slowly and accumulate in sediment and aquatic life	Legislative efforts to require DEQ to address PBTs have been attempted, but have not yet passed.
Fish consumption	Human health concern. Certain toxins (metals) bioaccumulate in fish tissue.	EPA/DEQ may increase the assumed quantity of fish consumed by a factor of 10, reducing effluent limits for bioaccumulative compounds accordingly. Could require higher level of treatment.

ATTACHMENT C

Blended treatment	Option for blended treatment may be disallowed	Neither city currently uses blended treatment
Lagoon leakage	DEQ guideline set forth limits on allowable leakage	Exceeding leakage criteria could result in requirement to line lagoons.
EPA reliability/redundancy requirements	Sets forth requirements for number and capacity of treatment units.	
Sanitary sewer overflows	All SSOs would be considered a non-compliance event	DEQ currently allows overflows during 5-year, 24-hour storm event. Design peak flows could increase, increasing costs.
Elimination of compliance schedules	Inclusion of compliance schedules in permits under litigation	Issue has delayed permit renewals state-wide. Unclear how time for making improvements would be granted.
Turbidity	Concern over the cumulative impact that effluent discharges have on river turbidity	Potential for numeric turbidity limits. Algae in lagoon effluent could make compliance with numeric limits difficult
New recycled water regulations	DEQ recently issued updated recycled water regulations	Regulations streamline recycled water use practices. Providing recycled water to irrigation districts is now allowed.

ATTACHMENT D



ALTERNATIVE "B"
Two Regional Wastewater Treatment Facilities

WASTEWATER FACILITIES PLAN
CITIES OF INDEPENDENCE AND MONMOUTH, OREGON
REGIONAL WASTEWATER TREATMENT ALTERNATIVES
Alternative B

REVISIONS: APPD.

DATE: NOV. 12, 2008
DESIGN: DMP
DRAFT: VASC
CHECKED:
REVISION NUMBER:

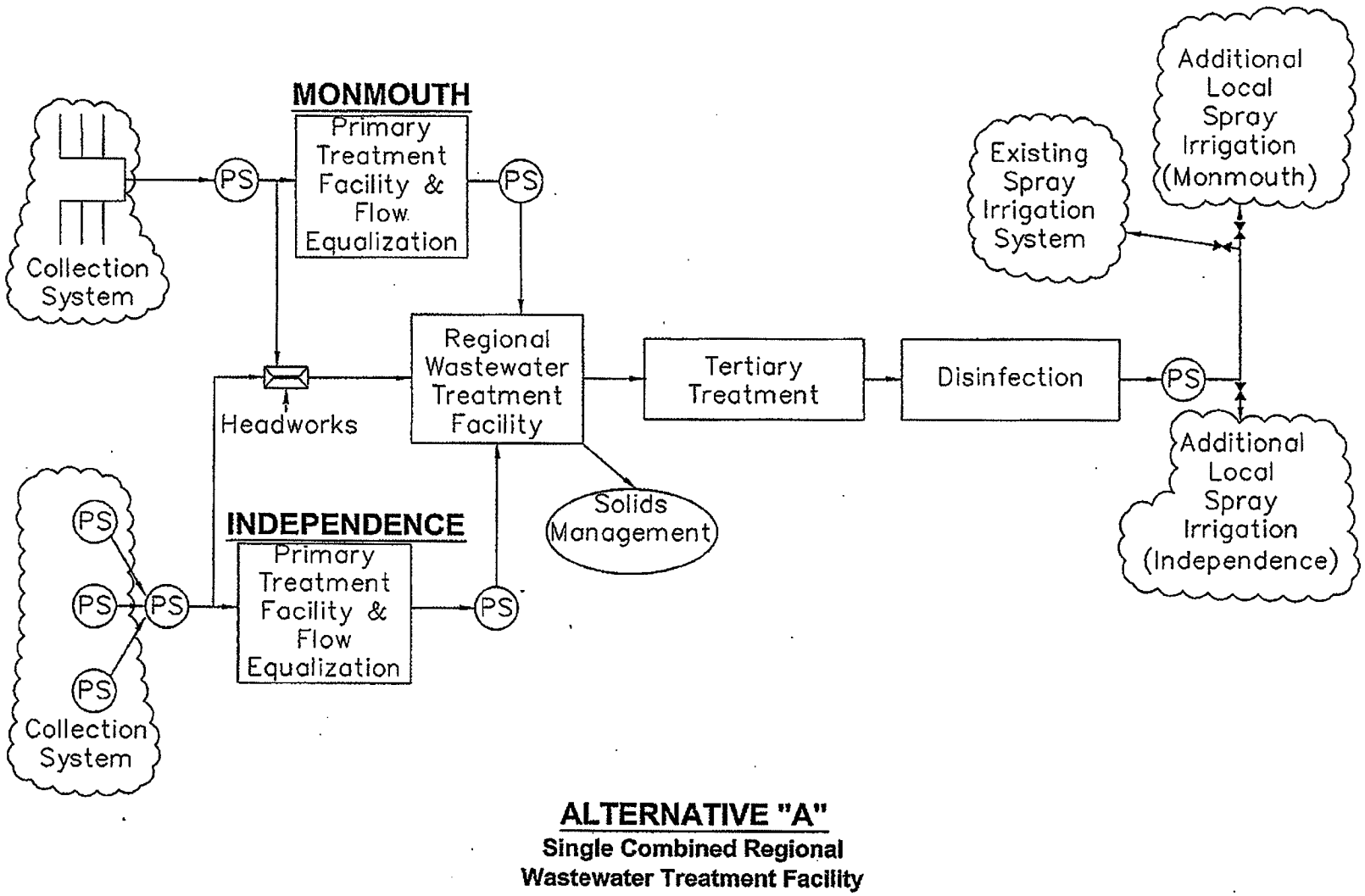
SCALE: No Scale

PROJECT NUMBER: INDP00000031

DRAWING FILE: Alternative B.dwg

SHEET NO. 2 OF 2

ATTACHMENT D



ALTERNATIVE "A"

Single Combined Regional Wastewater Treatment Facility

WASTEWATER FACILITIES PLAN
CITIES OF INDEPENDENCE AND MONMOUTH, OREGON
REGIONAL WASTEWATER TREATMENT ALTERNATIVES
Alternative A

REVISIONS: APPD.

DATE: NOV. 12, 2008
DESIGN: DEW
DRAWN: TASC
CHECKED:
REVISION NUMBER:

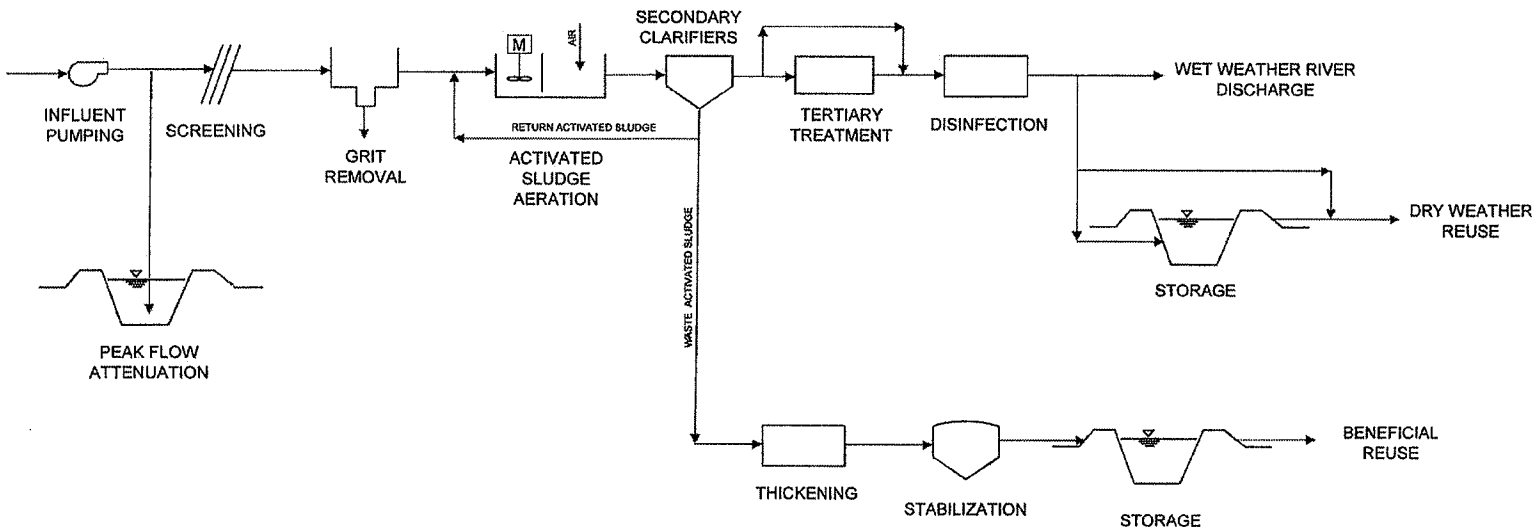
SCALE: No Scale

PROJECT NUMBER: INDP00000031

DRAWING FILE: Alternative A.dwg

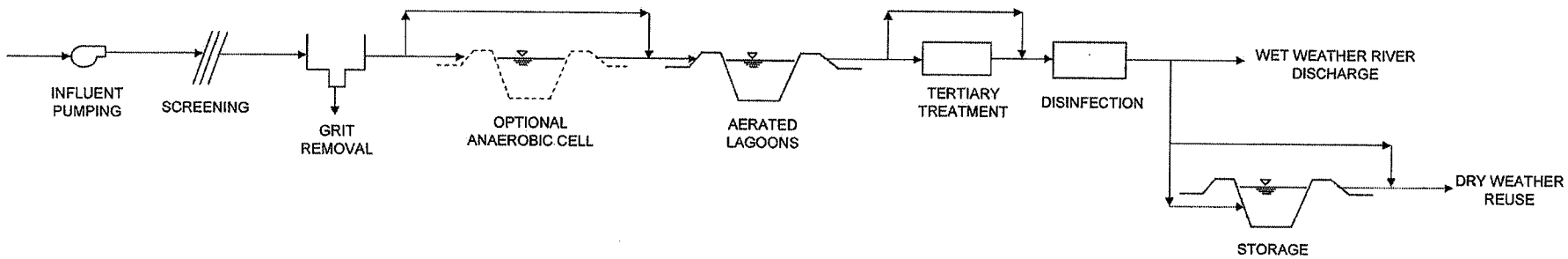
SHEET NO. 1 of 2

ATTACHMENT E



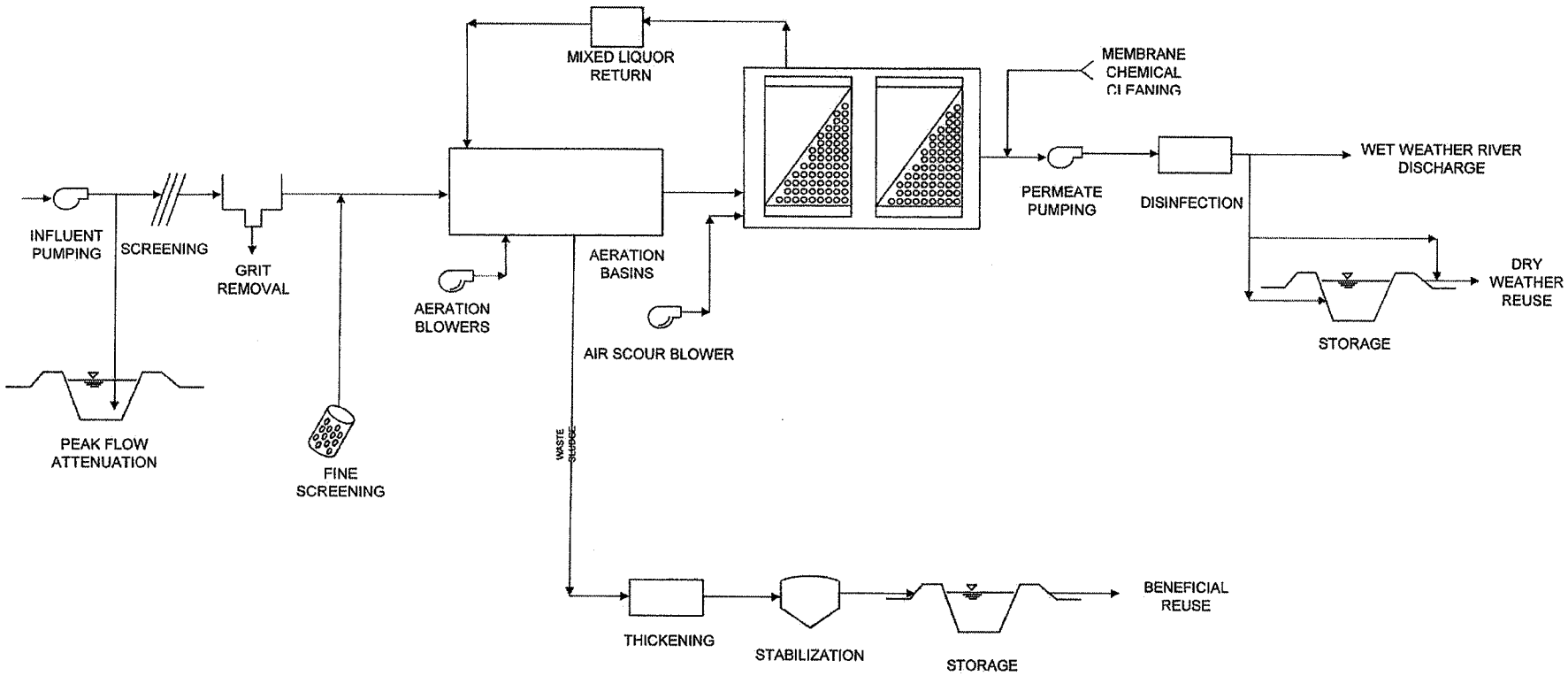
ACTIVATED SLUDGE

ATTACHMENT E



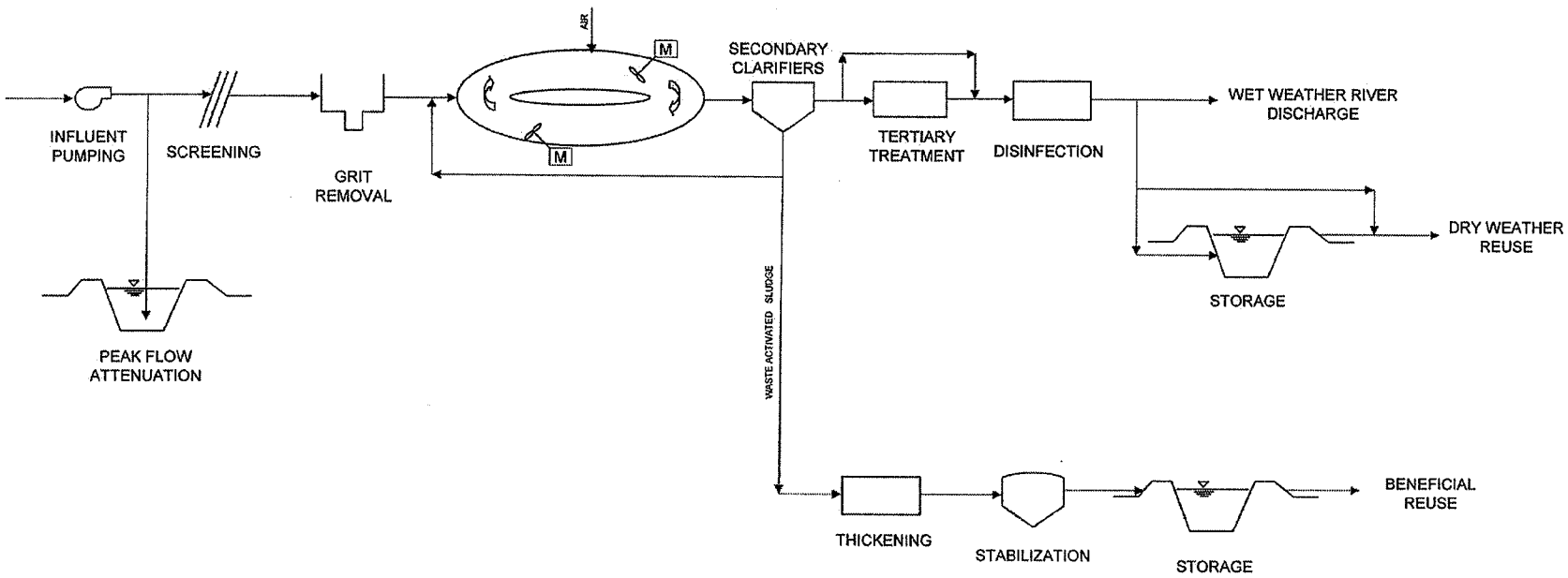
ADVANCED LAGOONS

ATTACHMENT E



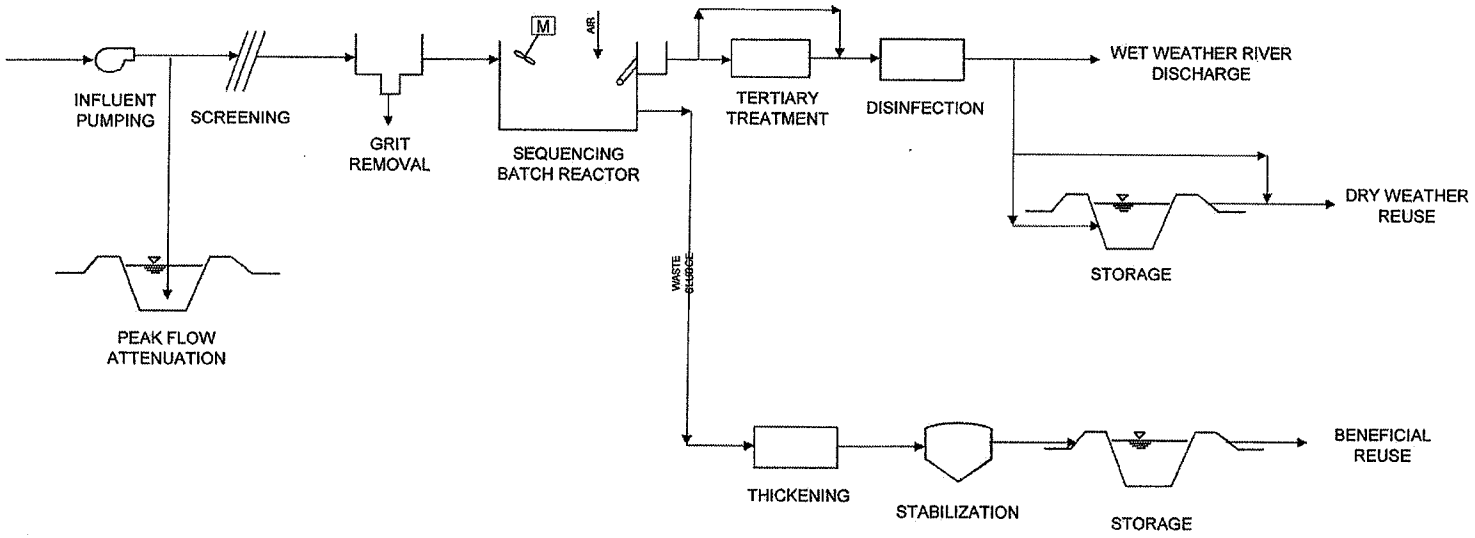
MEMBRANE BIOREACTOR

ATTACHMENT E



OXIDATION DITCH

ATTACHMENT E



SEQUENCING BATCH REACTOR

ATTACHMENT F

Option 1 - Individual Approach

Single Phase, Separate Projects

- Separate Activated Sludge Plants
- Separate Solids Processing Facilities
- Separate Effluent Reuse Facilities

Costs:

Independence = $47 Million

Monmouth = $48 Million

Option 2 - Partial Regionalization

Single Phase Project

- Separate Activated Sludge Plants
- Regionalized Solids Stabilization/Storage
- Regionalized Effluent Reuse Facilities

Costs:

Independence = $42 Million

Monmouth = $43 Million

Option 3 - Full Regional Approach

Single Phase Project

- Single Regional Plant
- Regionalized Solids Stabilization/Storage
- Regionalized Effluent Reuse Facilities

Costs:

Total Cost = $80 Million

Option 4 - Phased Regional Approach

Two-Phase Project

Phase I

- Individual Plants - Convert to Aerated Lagoons
- Regionalized Effluent Reuse Facilities
- Clean Accumulated Sludge from Lagoons
- Line Lagoons with membrane

Costs:

Phase I - 2010

Independence = $24 Million

Monmouth = $25 Million

Phase II

- Single Regional Plant
- Regionalized Solids Stabilization/Storage
- Regionalized Effluent Reuse Facilities
- Clean Aerated Lagoons of accumulated sludge

Costs:

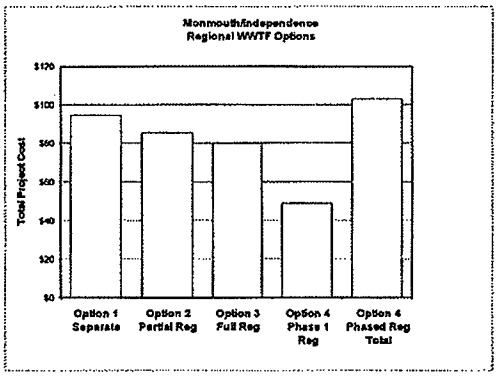
Phase II - 2020

Total Cost = $54 Million

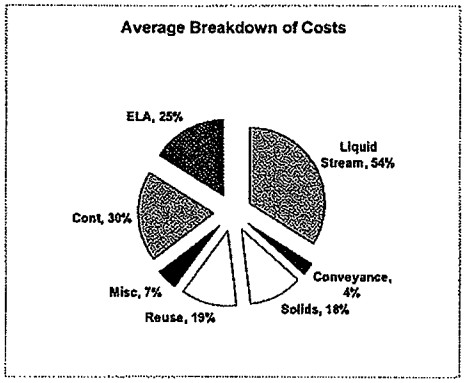
Total Phased Cost

Independence = $50 Million

Monmouth = $53 Million



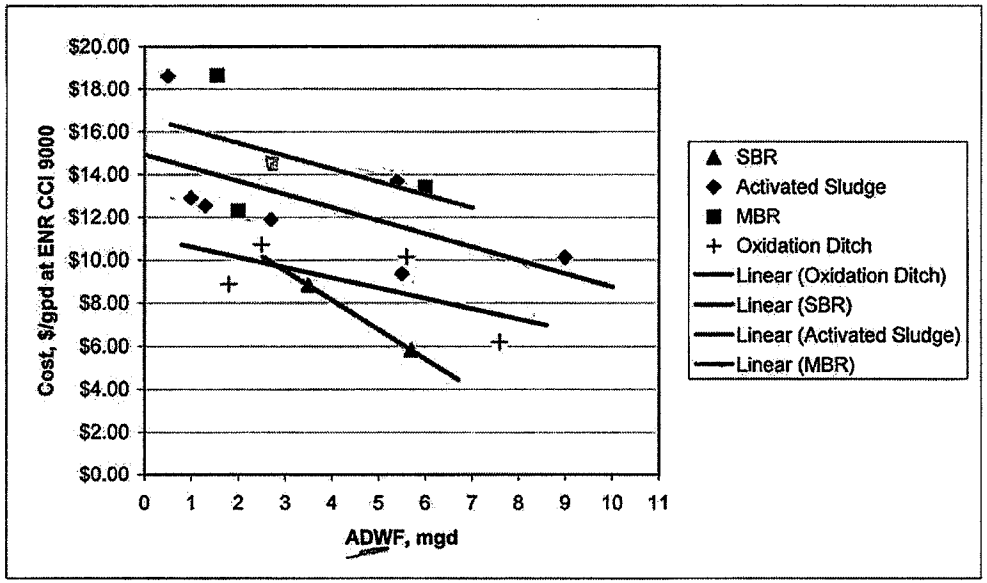
ATTACHMENT F



ATTACHMENT G

ADWF, mgd	Type/City	Unit Cost, $/gpd	Cost
	SBR		
3.5	Chehalis WA	$8.83	$30,900,000
5.7	Clear Lake, IA	$5.82	$33,200,000
	Activated Sludge		
2.7	Centralia WA	$11.89	$32,100,000
1	Reedsport OR	$12.90	$12,900,000
1.3	Florence OR	$12.54	$16,300,000
0.5	Cave Junction OR	$18.60	$9,300,000
9	Vacaville CA	$10.12	$91,100,000
5.4	El Dorado CA	$13.69	$73,900,000
5.5	Soledad CA	$9	$51,400,000
	MBR		
2	Lott WA	$12.30	$24,600,000
1.55	Blaine WA	$18.65	$28,900,000
6	Alderwood WA	$13.40	$80,400,000
	Oxidation Ditch		
1.8	Myrtle Creek	$8.89	$16,000,000
5.6	McMinnville OR	$10.14	$56,800,000
2.5	Mesa Co. CO	$10.72	$26,800,000
7.6	Willmar MN	$6.20	$47,100,000

Red = includes solids improvements



APPENDIX F

Reuse Alternatives

Whitaker Summary of Combined Reuse Analysis
Kennedy/Jenks Memorandum
GHD Review Memorandum

MONMOUTH/INDEPENDENCE JOINT EFFLUENT RECLAMATION PROJECT

DRAFT FEASIBILITY STUDY

September 2004

WHITAKER ENGINEERING INC.

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Monmouth-Independence Joint Effluent Reuse Project Feasibility Study

I. INTRODUCTION

A. Purpose and Scope – The purpose of this report is to determine the long-term (20-year) feasibility of a joint effluent reuse project between the Cities of Independence and Monmouth that would combine and transfer treated effluent from two wastewater treatment facilities to area farmland for beneficial reuse. In addition, this report will provide recommendations for facility improvements to provide reliable operation for a 20-year period. Project feasibility will be determined based on the following parameters:

- projected wastewater flows and characteristics,
- capacity of affected facilities at the Monmouth WWTP,
- irrigation site availability, and
- crop irrigation requirements.

B. Summary of Findings

1. Effluent Suitability

The Independence effluent is very similar to that of the City of Monmouth and no adverse effects from combining these two waters is expected.

2. Impact on Existing Facilities

Addition of Independence effluent will require modification to existing facilities to accommodate the increased loading.

3. Impact on Reuse Site Availability

Site #1 currently does not have adequate capacity to satisfy the surplus effluent generated by both communities through 2021. However, two additional sites (Sites 2 & 3) that are currently under DEQ review plus a fourth site will provide more land than needed to utilize projected effluent quantities. Final permitting of Sites 2 & 3 is expected during 2004 and Site 4 is expected to obtain approval during 2005.

4. Financial Impacts

Initial capital investment by the City of Independence will include the following:

- $210,000 for capacity improvements to the Monmouth facilities,
- $510,000 for a transfer pump station and pipeline to the Monmouth WWTP, and
- Compensation to landowner for contracting to irrigate required effluent amounts (estimated at $200,000).

In addition, Independence will incur a cost associated with reimbursing the City of Monmouth for a portion of the facilities already constructed as well as additional O&M costs.

For the purpose of this report, reimbursement to Monmouth was chosen to be in the form of a rate per million gallons of effluent transferred. The selected rate was adjusted so that the 20-year present worth value of all improvements by both communities is proportional to the anticipated effluent quantities discharged to irrigation from each community. The resulting rate is $3,270 per Mgal. If this cost were to be funded solely through sewer rate increase, the estimated impact is $5-$6 per month.

Other forms of reimbursement may be available, such as partial buy-in to the Monmouth facilities.

Both communities will have costs associated with other improvements not specifically required by the increased effluent addition. This report does not address these other costs.

The following Table 1 summarizes the magnitude of improvements required for Monmouth with and without the Independence effluent addition.

Table 1 – Magnitude of Anticipated Improvements to Unit Processes (Monmouth)

Unit Process or Facility	Improvements Required for Monmouth Alone	Improvements Required for Combined Flows
Facultative Lagoons	Major (2006)	Major (2006)
Chlorine Contact Basin	None	None
Chlorine Feed System	None	Major (2006)
Effluent Irrigation Pump	Minor (2016)	Major (2006)
Automatic Strainer	None	Major (2006)
Irrigation Pump Station Structural	None	Major (2006)
Transmission Pipeline	None	None
Booster Pump Station #1	Possible Major (2016)	Major (2006)

Minor - 2005 = minor improvements required in 2005
Major - 2005 = major improvements required in 2005
None = No Improvements Required

II. EXISTING CONDITIONS

A. Existing Wastewater Characteristics – Analytical results for effluent sampled from Monmouth and Independence are reported in Tables A-1 and A-2 of Appendix A. Table A-1 summarizes previous analytical testing on City of Monmouth effluent. Table A-2 presents recent analytical testing results for City of Independence effluent. Based on an examination of this data, there does not appear to be any constituent that is likely to limit the reuse of these effluents for agricultural purposes. In fact, most constituents are within the limits prescribed by EPA for safe drinking water.

B. Existing Unit Process Capacity – The unit capacity of various processes is included in Table 2. Only the capacity of unit process affected by the proposed addition of Independence effluent are considered here.

Table 2 – Unit Process Capacity

Unit Process	Parameter	Capacity
Monmouth Pond #3		
	HDT, days	28
	BOD5, #/d	460
CCB (9 ft deep, 60 min HRT)	Flow, mgd	1.8
CCB (9 ft deep, 20 min HRT)	Flow, mgd	5.4
Irrigation Pump	Flow, gpm	1,350
Automatic Strainer	Flow, gpm	1,700
Pipeline	Flow, gpm	2,900
Booster Station #1	Flow, gpm	1,200
Operating Capacity	Flow, gpm	1,085

C. Operating Capacity – The instantaneous flow capacity of the existing reclaimed effluent facilities is limited by capacity at the Irrigation Pump Station. As shown in Table 1, the irrigation pump has a capacity of 1,350 gpm. However, the strainer uses approximately 150 gpm during backwash, leaving about 1,200 gpm of sustained flow available to Site #1. Also, each irrigation zone requires a higher flow rate for several minutes each time it starts in order to fill the submains and laterals and bring the zone up to pressure. The initial fill rate is limited to 1,200 gpm by the irrigation pump capacity. Experience has shown that a sustained flow of 1,085 gpm is the upper limit that can be supplied without exceeding the irrigation pump capacity during this initial fill cycle.

D. Existing Reuse Site(s) – To date, one reuse site has been permitted for beneficial reuse of Monmouth WWTP effluent. This property is located on the west side of Helmick Road approximately 2 miles south of Monmouth and consists of one tax lot totaling 291 gross acres. The site is currently planted in hybrid poplars (57 acres), ponderosa pine (11 acres), black walnut (49 acres), annual ryegrass (69 acres), and perennial fescue (68 acres). Only the tree plantations are currently under irrigation. This site, when fully developed as a tree plantation, will utilize up to 120 Mgal per year. While this will satisfy Monmouth's need for beneficial use, it will not provide adequate demand to utilize surplus effluent from both communities.

Two additional site reuse plans are under review by DEQ and one additional site reuse plan is expected to be submitted to DEQ during 2004. These sites will be discussed further in the following sections.

III. FUTURE CONDITIONS

A. Projected Flows – Table 3 provides estimated surplus wastewater quantities for Monmouth, Independence, and the two cities combined through 2021. These tables provide projections for surplus quantities for three scenarios:

- Design Case #1 – This scenario assumes that population will grow at an annual rate 0.5% higher than current projections and precipitation/evaporation will be

similar to a wet summer with a 10-year recurrence interval. Under this scenario, the crop water demand is minimized while surplus effluent quantities are maximized. *This condition will determine the minimum number of acres that should be permitted and available for irrigation with 1-2 years advance notice.*

- Design Case #2 – This scenario assumes that population will grow as projected, based on recent trends, and precipitation/evaporation will be similar to a wet summer with a 10-year recurrence interval. *This condition will determine the actual crop water demand that must be developed each year.*
- Design Case #3 – This scenario assumes that population will grow at an annual rate 0.5% lower than current projections and precipitation/evaporation will be similar to a drought condition with a 10-yr-recurrence interval. Under this scenario, the crop need would be highest while the available effluent quantities would be lowest. *This condition will determine the additional agricultural storage required to minimize crop impact during dry years.*

Appendix B presents water budget information for the three design cases discussed above. The projected quantities are based on historical population and flow data for the two communities. The projected surplus quantities assume the two treatment facilities will continue to utilize the existing lagoons for treating or storing effluent. In calculating the surplus quantities shown in these tables, it has been assumed that both facilities will require mechanical aeration at some time in the future and this will limit the minimum water depth to 4.5 feet.

Table 3 – Projected June through October Surplus Effluent Quantities (Mgal) for Monmouth and Independence

	Monmouth			Independence			Combined		
Year	Low MGal	Avg MGal	High MGal	Low MGal	Avg MGal	High MGal	Low MGal	Avg MGal	High MGal
2004	30	44	44	32	46	46	61	91	91
2006	34	49	50	36	51	52	70	100	102
2011	46	61	66	47	62	66	94	123	132
2016	60	75	83	60	75	83	120	149	167
2021	75	90	104	74	89	103	149	179	206

IV. CAPACITY ANALYSIS

A. Reuse Site Availability

1. Site 1 – This 300 acre permitted site is has the capacity to receive up to 120 Mgal per irrigation season and is currently under a long-term contract with the City of Monmouth to receive the city's excess wastewater.

2. Site 2 – This 154-acre site is currently under review by the Department of Environmental Quality. It is located east of Site 1 approximately 1.75

miles south of Monmouth and is transected by the existing 16-inch reclaimed effluent transmission pipeline. The estimated potential capacity of this site is between 35 and 85 Mgal, depending on crop selection.

3. Site 3 – This 300-acre site is currently under review by the Department of Environmental Quality. It is located east of Site 2 approximately 1.75 miles south of Monmouth and is transected by the existing 16-inch reclaimed effluent transmission pipeline. The estimated potential capacity of this site is between 69 and 166 Mgal, depending on crop selection.

4. Site 4 – This 152-acre site is currently in the permitting stage and is expected to be submitted to the Department of Environmental Quality this year. It is located north of Site 3 approximately 0.7 miles south of Monmouth and is transected by the existing 16-inch reclaimed effluent transmission pipeline. The estimated potential capacity of this site is between 35 and 84 Mgal, depending on crop selection.

B. Water Budget Analysis

Based on the water budget analyses presented in Appendix B for the three design cases discussed in Section III above, the following conclusions can be made:

1. Assuming Sites 2, 3, and 4 are permitted, a total of 830 acres will be available for beneficial reuse. This is adequate to produce a demand for anticipated surplus effluent. At 2021 maximum surplus effluent production rates of 206 Mgal, the average irrigation application on all four sites would be approximately 9 inches.

2. Under projected average conditions (Design Case #2), the existing contracted acreage at Site #1, while adequate for Monmouth, is not adequate to utilize the projected volume of surplus effluent from both communities. Approximately 200 additional acres will need to be contracted, depending on the type of crops anticipated.

3. If reuse site acreage is contracted to accommodate Design Case #2, then an under-supply of effluent may exist during a dry year. This can be mitigated with on-site storage facilities at one or more irrigation sites. The estimated size of storage reservoir required to supply crops with optimum water during a dry year is 100 Mgal (300 ac-ft).

Sites 2, 3, and 4, if permitted, will add an additional 600 acres of land under production with reclaimed effluent. Irrigation demand from this land will exceed the supply of reclaimed wastewater from both communities through 2021. However, cropping choices may require additional storage.

Market conditions will influence future cropping choices. The City of Monmouth Effluent Reuse Plan includes water/nutrient uptake information for several selected crops that are currently being grown or have a good potential for selection in the future. Water uptake rates range from 19 inches per year for grass seed crops to 37 inches per year for mature poplar. With average June-October precipitation totaling 7 inches, the remaining plant needs range from 12 inches to 30 inches. A conservative design provides 80% of the consumptive needs of the crop through irrigation, forcing the plants to take the remaining 20% from the soil. This

maximizes the beneficial use of the reclaimed wastewater and minimizes deep percolation. **This approach results in typical annual irrigation applications between 10 and 24 inches.**

The City of Monmouth Effluent Reuse Plan also provides agronomic nutrient requirements for various crops. For each inch of reclaimed effluent applied, the nutrient loading is expected to be as follows:

- Nitrogen—2-5 lbs/acre
- Phosphorous—0.8-1.2 lbs/acre
- Potassium—2 lbs/acre

While most anticipated crops have nutrient requirements in excess of that provided in the reclaimed effluent, it is important to revise these numbers annually so that agricultural users can make adjustments can be made in the application of supplemental fertilizer.

C. Unit Process Capacity

The existing unit process have been evaluated based on their ability to meet the existing and future needs of the City of Monmouth as well as meeting the future needs of both Monmouth and Independence. This evaluation is presented in Appendix C.

The following design assumptions and criteria have been made or established in performing the capacity analysis in order to provide operational redundancy and flexibility to accommodate unforeseen conditions:

- 16-inch PVC transmission pipe, maximum design pressure: 165 psig
- 16-inch PVC transmission pipe, maximum operating pressure: 110 psig
- Chlorine contact time at average flow conditions: 60 minutes
- Chlorine contact time at peak flow conditions (10-yr event): 20 minutes
- Maximum irrigation schedule for dry/average conditions: 5d/wk, 12 hr/d
- Maximum irrigation schedule for peak conditions: 7d/wk, 12 hr/d
- Irrigation season is between May 15 and October 1
- Chlorine contact time in 16-inch PVC transmission pipe may be considered
- Independence effluent will be conveyed to Monmouth Pond #3
- Both treatment plants will eventually be converted to aerated lagoon system with the Monmouth WWTP requiring at least partial conversion prior to implementation of the proposed joint effluent reuse program.

These criteria have been compared to the minimum schedules required for current and future conditions to determine the feasibility of the proposed addition. The conclusions reached in the analysis presented in Appendix C are as follows:

1. The existing facilities will accommodate future flows through 2016 from Monmouth without requiring an irrigation schedule that exceeds established

criteria or restricting the location of the first point of use of reclaimed water (i.e. branch flow to WOU campus as STA 7+00).

2. The existing facilities will not accommodate Monmouth or combined flows through 2021.

3. Modest facility improvements to existing facilities will accommodate projected flows from Monmouth through 2021 without restricting the location of the first point of use of reclaimed water. Improvements required for this scenario would likely include the following:

   - New motor, impellers, and stage for existing effluent irrigation pump, and
   - New pump and filter at Booster Station #1.
   - Control system modifications

4. Major improvements to the capacity of the existing facilities will accommodate all future flows from Monmouth and additional flow from Independence through 2021. The required physical improvements would likely include the following:

   - Addition of second effluent irrigation pump and modification of existing pump/motor,
   - Addition of new automatic strainer and control panel,
   - Structural improvements to existing suspended pre-cast slabs,
   - Addition of third pump and filter at Booster Station #1 and/or addition of irrigation facilities at Sites 2, 3, and/or 4.
   - Control system modifications.

5. Addition of Independence effluent to the Monmouth WWTP will increase flow and decrease chlorine contact time. Since Monmouth uses a portion of the transmission pipeline to satisfy the chlorine contact time, the addition of Independence effluent may increase the length of pipe required to obtain the target contact time. This may have the effect of "passing by" potential reuse sites that could be developed within the City limits of Monmouth to decrease the summer demand for potable water. This disadvantage can be overcome by either an expanded Monmouth chlorine contact basin or by transferring chlorinated water from Independence WWTP directly into the Monmouth CCB.

V. PROPOSED FACILITY IMPROVEMENTS

Regardless of whether Independence effluent is transferred to the Monmouth WWTP, some improvements to the existing effluent reclamation facilities will eventually be required. However, addition of the Independence effluent will require immediate construction of more extensive facilities. Without the Independence effluent, it is expected that the existing Monmouth effluent reclamation facilities will

satisfy Monmouth's needs through 2016, when modest capacity improvements would be required. The following section addresses the facilities required at Independence if the two effluents are to be combined as well as the required future improvements to the Monmouth effluent reclamation facilities.

A. Independence Improvements – No improvements are anticipated for existing facilities at the Independence WWTP, however, a transfer pump station will be required to convey treated effluent from Pond #4 to the Monmouth WWTP. The anticipated design criteria for this pump station and force main are shown below:

Pump Station

1. Type: Self-priming or submersible, depending on preference and site constraints
2. Wet Well: Existing Chlorine Contact Basin
3. Pump Type: Constant speed, non-clog, centrifugal
4. [redacted]
5. Motor: 20 HP, 480V, 3 ph, 60 Hz
6. Level Control Type: Timer with low/high float Switch
7. Stand-By Power: None
8. Alarm Telemetry Type: Radio Modem
9. EPA Reliability Class: III

Force Main

10. Length: 8,200 ft
11. [redacted]
12. Material: PVC C-900
13. Profile: Flat
14. Discharge Location: Monmouth WWTP
15. Detention Time: 45 minutes

B. Improvements required for Monmouth effluent alone – The following capacity related improvements to the Monmouth Effluent Reuse Facilities will be required to serve only Monmouth:

1. Disinfection System – The following improvements to the existing disinfection system will be required

   a) Chlorine contact basin improvements – No improvements required.

   b) Chlorine feed equipment – No improvements required.

2. Effluent Irrigation Pump Station – The following improvements to the existing pump station will be required:

   a) The existing pump will require a new motor (and possibly a new adjustable speed drive), impellers, and a fourth stage, to obtain the following design points:

Design Point	Flow (gpm)	TDH (ft)
#1	1500	230
#2	1300	230
#3	330	230

The total discharge head required will depend on the pressure required at the anticipated point(s) of use. The final design criteria should be coordinated with anticipated irrigation needs to minimize the need for booster pumping at remote irrigation sites.

b) Effluent Strainer – No improvements required.

c) Structural Improvements – No improvements required.

3. Transmission Pipeline – No improvements are required.

4. Booster Station #1 – A third pump and filter may be required at Booster Station #1 to accommodate anticipated peak flows, depending on the water use of additional irrigation sites.

5. Irrigation Site Acreage – The existing Site #1 acreage will accommodate the needs of the City of Monmouth.

Improvements required for combined Monmouth/Independence effluent – The following improvements to the Monmouth Effluent Reuse Facilities will be required if addition of Independence effluent occurs:

1. Disinfection System – The following improvements to the existing disinfection system will be required

   a) Chlorine contact basin Improvements – No improvements required unless there will be an anticipated use of reclaimed effluent at pipeline stations less than 70+00.

   b) Chlorine feed system improvements – Chlorine feed capacity will need to be increased at peak flow for chlorine dosage rates in excess of 17 mg/l.

2. Effluent Irrigation Pump Station – The following improvements to the existing pump station will be required:

   a) A second pump and adjustable speed drive will be required to increase the total pump station capacity to 3,000 gpm at 230 ft TDH. The actual total dynamic head required will depend on the pressure required at the anticipated point(s) of use. The final design criteria should be coordinated with anticipated irrigation needs to minimize the need for booster pumping at remote irrigation sites.

   b) Effluent Strainer – A second effluent strainer and control panel similar to the existing strainer and control panel will be required.

c) Structural Improvements – The existing pre-cast pump and strainer platform will need to be extended to the east to accommodate the new equipment.

3. Transmission Pipeline – No improvements are required.

4. Booster Station #1 – A third pump and filter may be required at Booster Station #1 to accommodate anticipated peak flows, depending on the water use of additional irrigation sites.

5. Irrigation Site Acreage – As discussed previously, additional sites will be required for generating an irrigation demand in excess of the surplus storage. At the time of this report, two additional sites were under review by DEQ and a third site was under development. Assuming these sites will be permitted, no further sites will be required to satisfy the anticipated flows from both communities.

Appendix A
Wastewater Characteristics

Table A-1
Monmouth Wastewater Composition

Analyte	Reporting Limit	Units[1]	Concentration[2]
General Chemistry			
TSS		mg/l	10
BOD		mg/l	10
pH			7.4
Alkalinity	0.5	mg/l	166.0
Chloride	1.0/0.1	mg/l	33.7
Fluoride	1.0/0.1	mg/l	0.96
TKN	2.0	mg/l	10.2
Ammonia	0.1	mg/l	1.6
Nitrate	0.01	mg/l	0.2
Nitrite	0.01	mg/l	0.4
TDS	1.0/5.0	mg/l	276.0
P	0.20	mg/l	4.2
Sulfate	1.0/0.1	mg/l	21
TOC	10.5/5.0	mg/l	36.7
Metals			
Antimony	5.0	µg/l	5
Arsenic	10.0	µg/l	10
Barium	25.0	µg/l	25.3
Beryllium	4.0	µg/l	4
Boron	275	µg/l	275
Cadmium	5.0	µg/l	5
Calcium	500	µg/l	35450
Chromium	10.0	µg/l	10
Cobalt	10.0	µg/l	10
Copper	10.0	µg/l	19.25
Lead	3.0	µg/l	3
Magnesium	500	µg/l	12850
Nickel	20.0	µg/l	20
Potassium	2000	µg/l	8785
Selenium	7.0	µg/l	7
Silicon	500	µg/l	11600
Silver	10.0	µg/l	10
Sodium	1000	µg/l	42950
Thallium	10.0	µg/l	10
Tin	25.0	µg/l	25
Vanadium	25.0	µg/l	25
Zinc	20.0	µg/l	27
Mercury	0.03	µg/l	0.03

Table A-1
Monmouth Wastewater Composition

Analyte	Reporting Limit	Units[1]	Concentration[2]
Disinfection Byproducts			
Chloroacetic acid	1.3	µg/l	1.3
Bromoacetic acid	0.7	µg/l	0.7
Dichloroacetic acid	0.7	µg/l	5.5
Trichloroacetic acid	0.7	µg/l	2.1
Bromochloroacetic acid	0.7	µg/l	0.7
Dibromoacetic acid	0.7	µg/l	0.7
Chloroform	0.5	µg/l	1.7
Bromodichloromethane	0.5	µg/l	0.5
Dibromochloromethane	0.5	µg/l	0.5
Bromoform	0.5	µg/l	0.5
Total Organic Halides	50	µg/l	76.2

Notes:
[1]mg/l = ppm, µg/l = ppb
[2]If analyte was below detection limits, the concentration was assumed to be equal to the detection limit.

Table A-2
Independence Wastewater Composition

Analyte	Reporting Limit	Units[1]	Concentration[2]
General Chemistry			
TSS		mg/l	18.9
BOD		mg/l	6.6
pH			7.69
Alkalinity		mg/l	122
Chloride		mg/l	43
Fluoride	0.1	mg/l	0.1
TKN		mg/l	8.9
Ammonia		mg/l	4.9
Nitrate	0.1	mg/l	0.6
Nitrite		mg/l	0.1
TDS		mg/l	210
P		mg/l	1.39
Sulfate		Mg/l	13.9
TOC		Mg/l	26
Metals			
Antimony	5.0	µg/l	5^{U}
Arsenic	2.0	µg/l	2^{U}
Barium	100	µg/l	100^{U}
Beryllium	0.2	µg/l	0.2^{U}
Boron		µg/l	160
Cadmium	2.0	µg/l	2^{U}
Calcium		µg/l	9.2
Chromium	10	µg/l	10^{U}
Copper	10	µg/l	10^{U}
Lead	2	µg/l	2^{U}
Magnesium		µg/l	7200
Nickel	10	µg/l	10^{U}
Potassium		µg/l	5400
Selenium	2	µg/l	2^{U}
Silica		µg/l	24500
Silver	5	µg/l	5^{U}
Sodium		µg/l	25100
Thallium	1	µg/l	1^{U}
Tin	50	µg/l	50^{U}
Vanadium	5	µg/l	5^{U}
Zinc		µg/l	110
Mercury	1	µg/l	1^{U}

Table A-2
Independence Wastewater Composition

Analyte	Reporting Limit	Units[1]	Concentration[2]
Disinfection Byproducts			
Chloroacetic acid		µg/l	1.9
Bromoacetic acid	1.0	µg/l	1 U
Dichloroacetic acid		µg/l	8.3
Trichloroacetic acid		µg/l	2.9
Bromochloroacetic acid	1.0	µg/l	1 U
Dibromoacetic acid	1.0	µg/l	1 U
Chloroform		µg/l	5.0
Bromodichloromethane	0.5	µg/l	0.5 U
Dibromochloromethane	0.5	µg/l	0.5 U
Bromoform	0.5	µg/l	0.5 U
HAA5	1.0	MCL	60

Notes:
[1]mg/l = ppm, µg/l = ppb
[2]If analyte was below detection limits, the concentration was assumed to be equal to the detection limit.

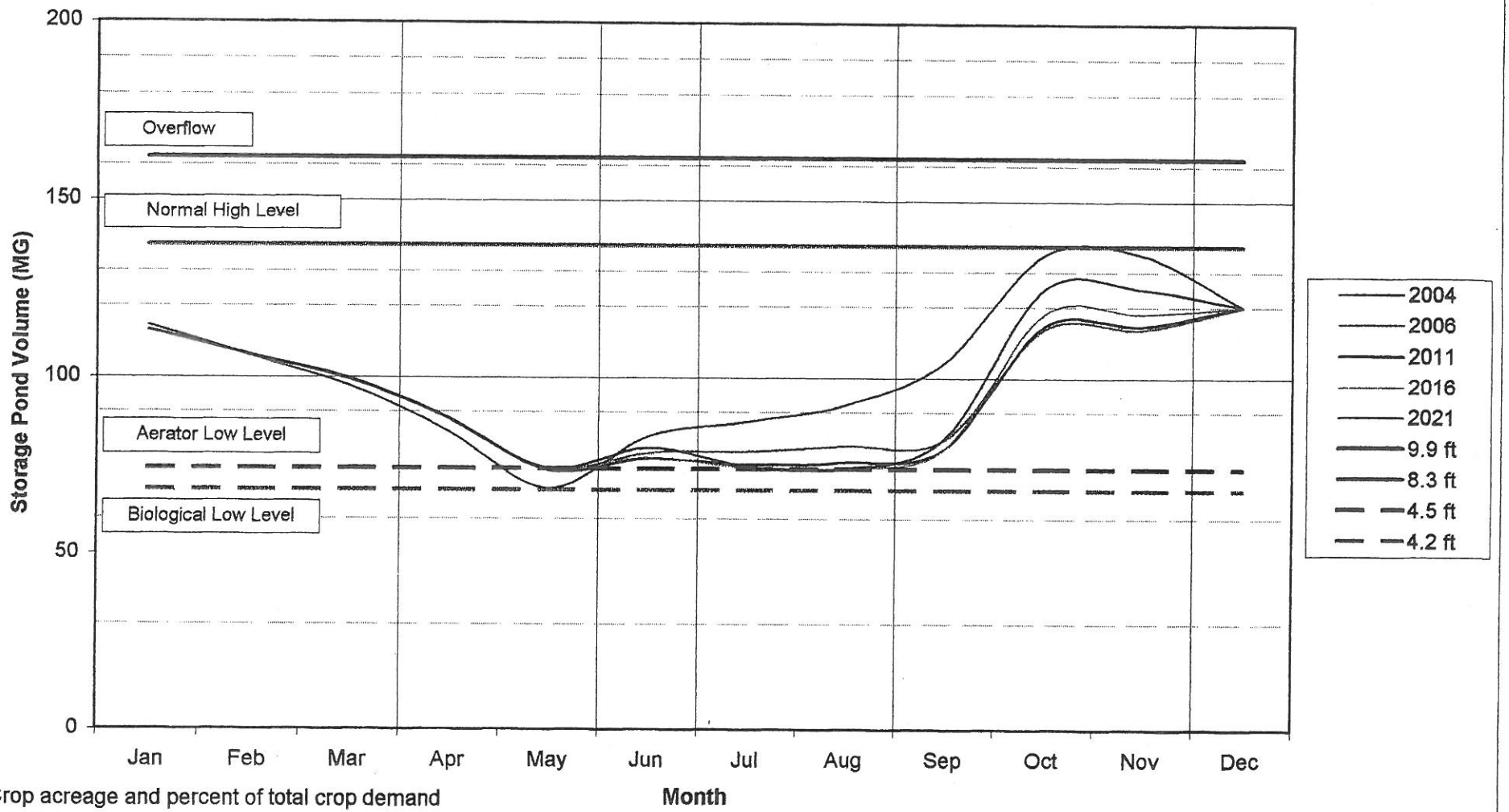
Appendix B

Design Cases – Lagoon Water Balances

Design Case #1

This design case assumes that both communities will grow at a rate approximately 0.5% higher than projected, precipitation will be at a ten year high, and evaporation will be 94% of average (corresponding with 10-yr precipitation).

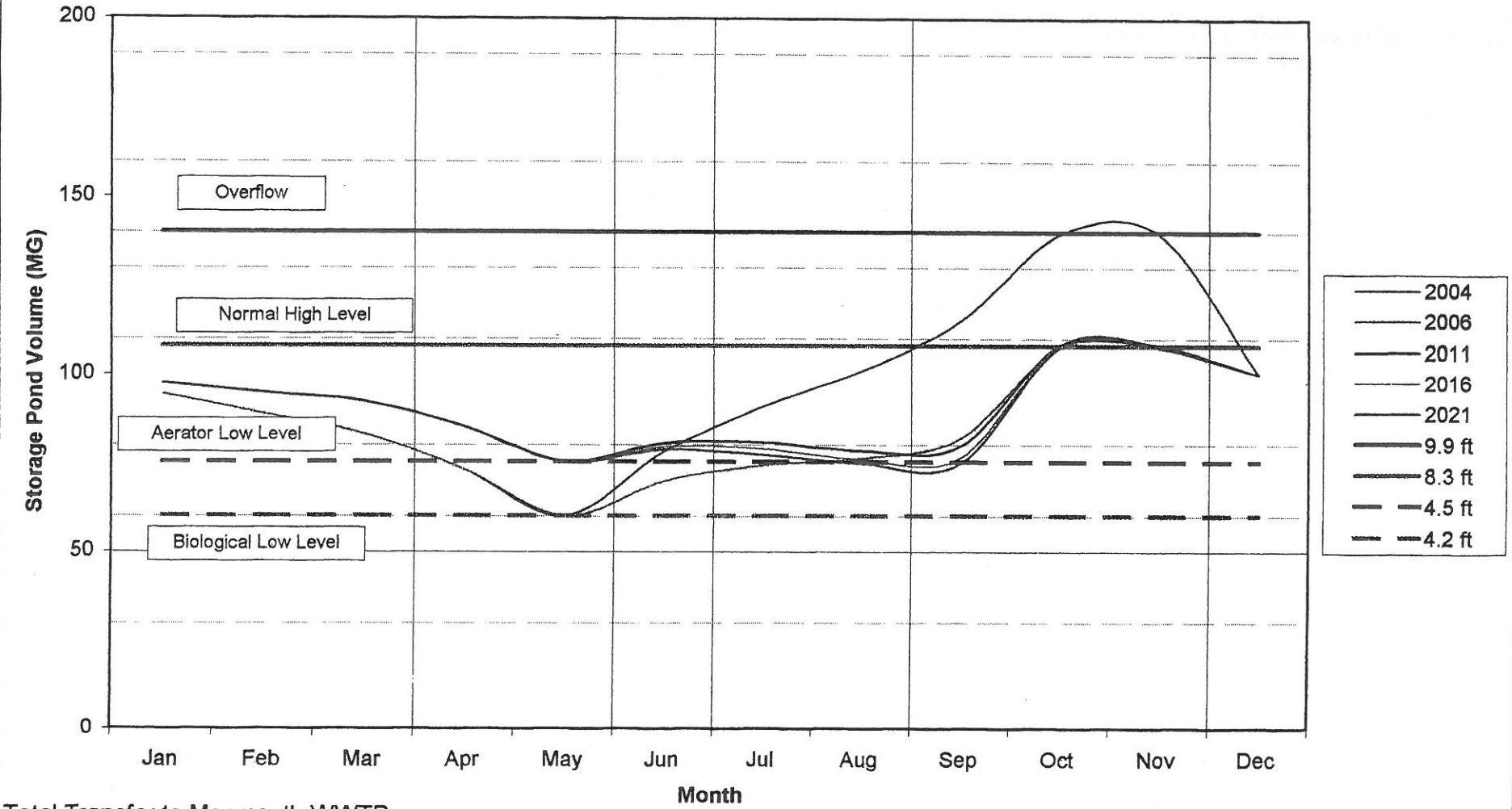
The crop acreage required by this design case is not the actual amount that must be planted and irrigated on a given year. Instead, it is the amount that must be available to be planted and irrigated (i.e. approved Site Reuse Plan) with reasonable advance notice (1-2 yrs). By revising the population projections (and subsequently the crop acreage requirements) on a 5-year interval, the crop acreages for this design case do not have to be developed until actual census information reflects growth higher than normal.



Crop acreage and percent of total crop demand satisfied:

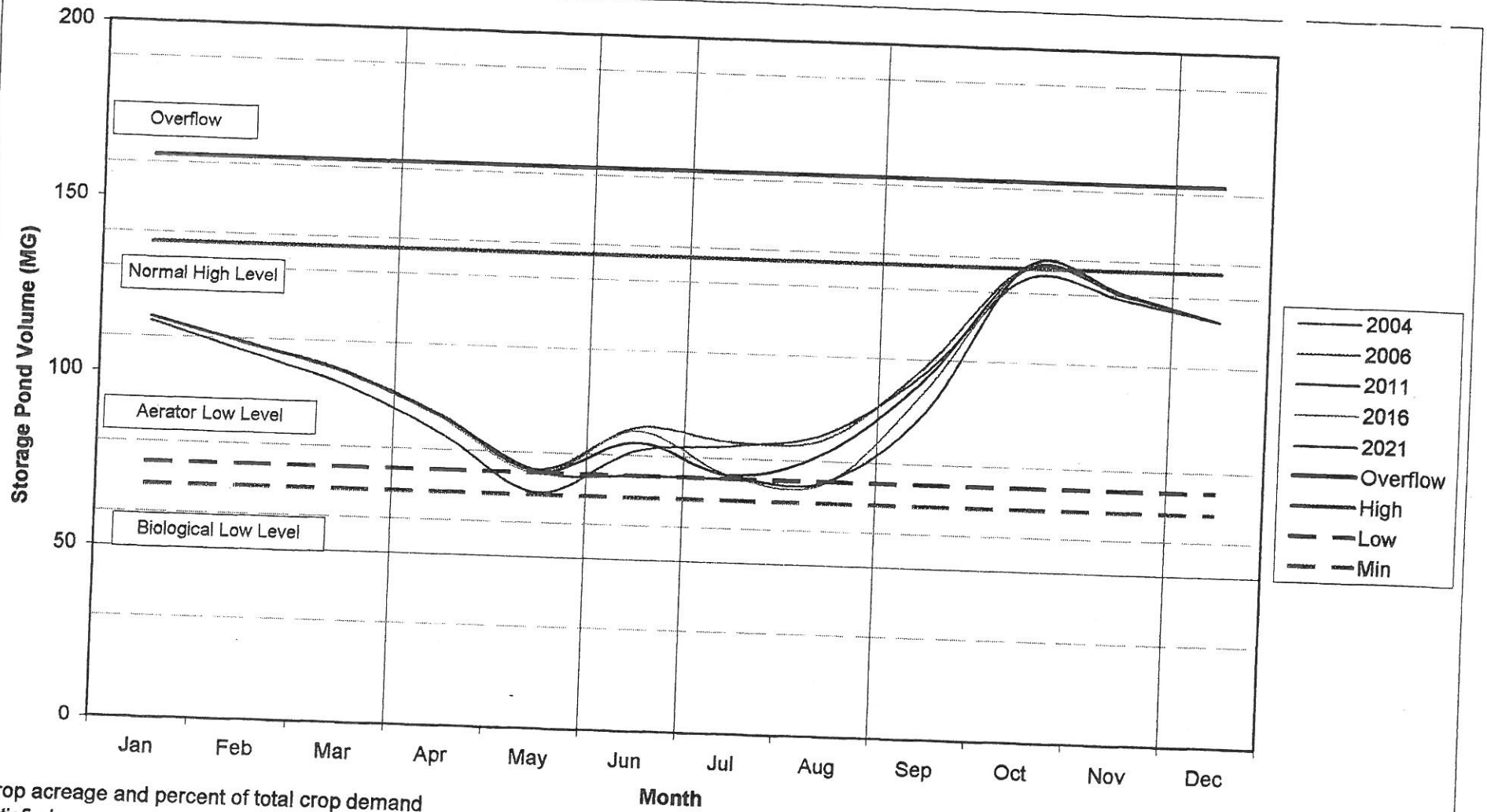
1) 2004: 117 ac trees @ 69%, 127 ac grass @ 8%
2) 2006: 185 ac trees @ 71%, 644 ac grass @ 8%
3) 2011: 185 ac trees @ 66%, 644 ac grass @ 8%
4) 2016: 244 ac trees @ 60%, 585 ac grass @ 8%
5) 2021: 244 ac trees @ 71%, 585 ac grass @ 8%

Figure B-1
Monmouth WWTP
Storage Pond Water Balance
Monmouth Flows Only - Design Case #1



Total Transfer to Monmouth WWTP:
2004: 0 MG
2006: 37 MG
2011: 50 MG
2016: 83 MG
2021: 103 MG

Figure B-2
Independence WWTP
Storage Pond Water Balance
Design Case #1



Crop acreage and percent of total crop demand satisfied:

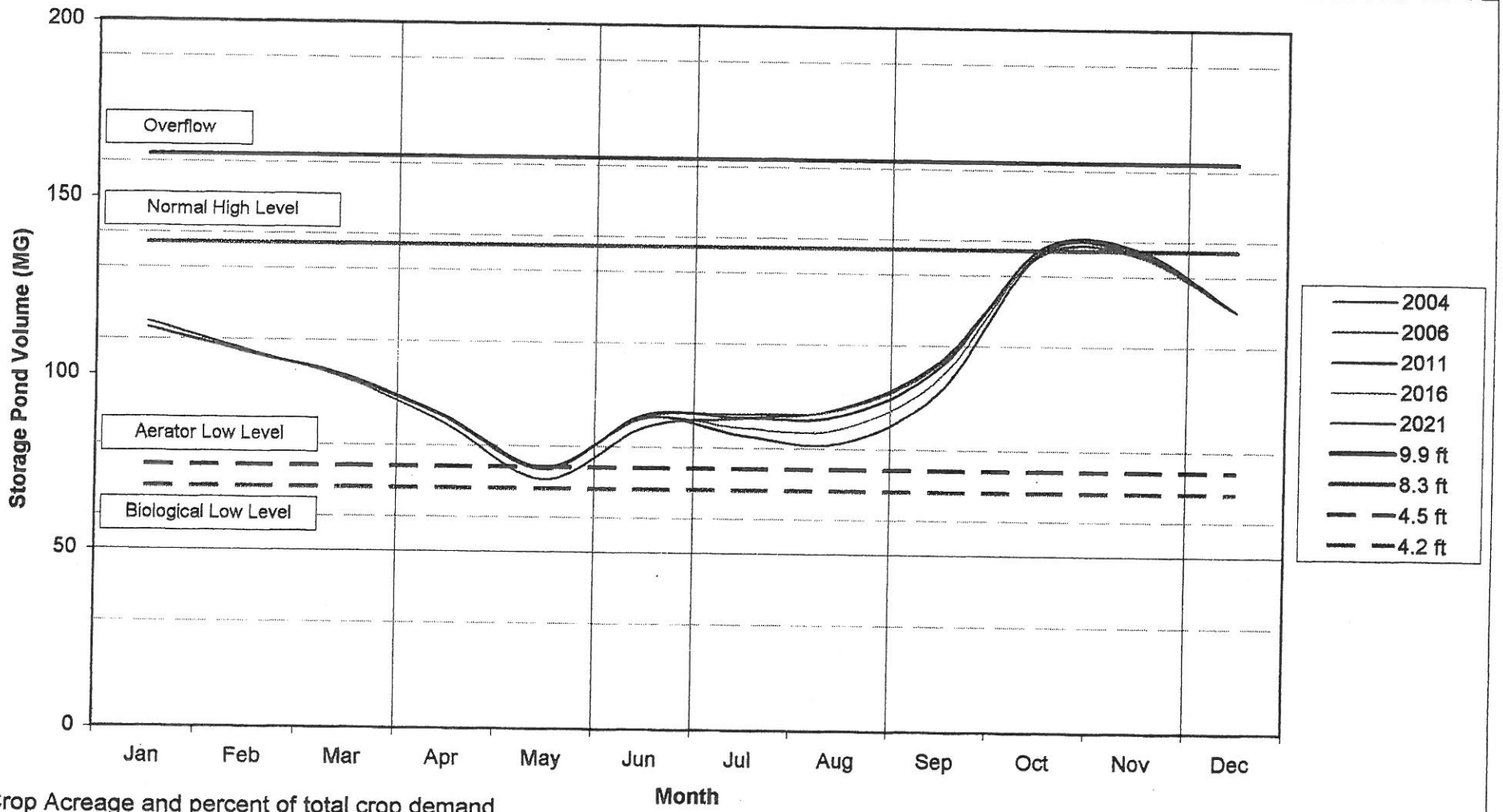
1) 2004: 117 ac trees @ 87%, 127 ac grass @ 7%
2) 2006: 207 ac trees @ 87%, 250 ac grass @ 7%
3) 2011: 310 ac trees @ 78%, 519 ac grass @ 7%
4) 2016: 360 ac trees @ 86%, 469 ac grass @ 7%
5) 2021: 650 ac trees @ 55%, 119 ac grass @ 7%

Figure B-3
Monmouth WWTP
Storage Pond Water Balance
Combined Flows - Design Case #1

Design Case #2

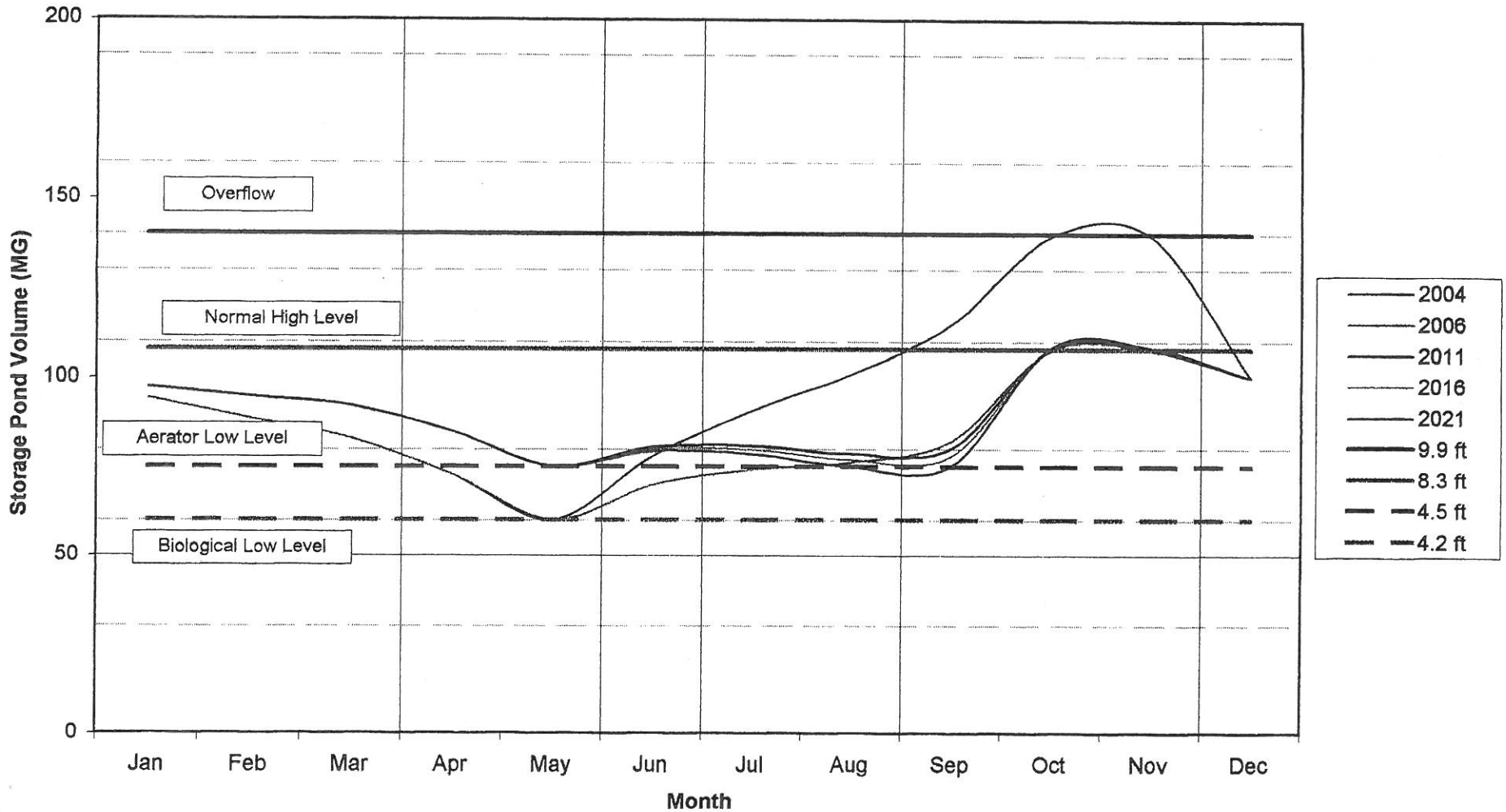
This design case assumes that both communities will grow at a rates that are similar to recent growth trends, precipitation will be at a ten year high, and evaporation will be 94% of average (corresponding with 10-yr precipitation).

The crop acreage required by this design case is the crop acreage that must be planted and available to receive irrigation on any given year. Because predicting a "wet season" is difficult, the City and landowner must always approach the irrigation season with the assumption that it will be a wet year (precipitation at a ten year high). In doing so, the City should be protected from overflow or early discharge during a wet season with a 10% chance of recurrence.



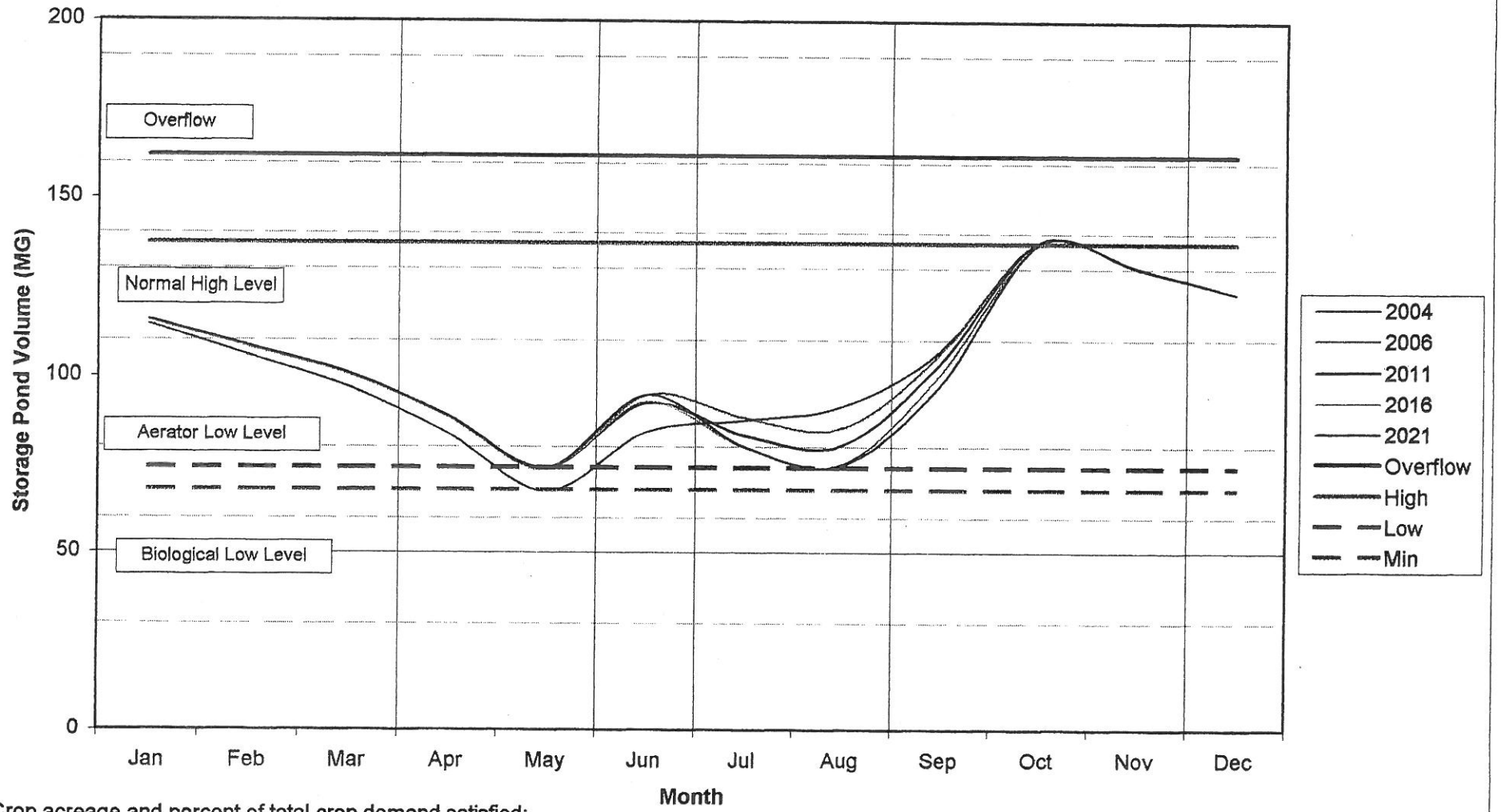
Crop Acreage and percent of total crop demand satisfied:
2004: 117 ac trees @ 72%
2006: 137 ac trees @ 72% (incl. 20 ac of 2-yr poplar)
2011: 197 ac trees @ 72% (incl. 80 ac of 1-yr poplar)
2016: 185 ac trees @ 72%
2021: 216 ac trees @ 72%

Figure B-4
Monmouth WWTP
Storage Pond Water Balance
Design Case #2



Total Transfer to Monmouth WWTP:
2004: 0 MG
2006: 36 MG
2011: 62 MG
2016: 75 MG
2021: 89 MG

Figure B-5
Independence WWTP
Storage Pond Water Balance
Design Case #2



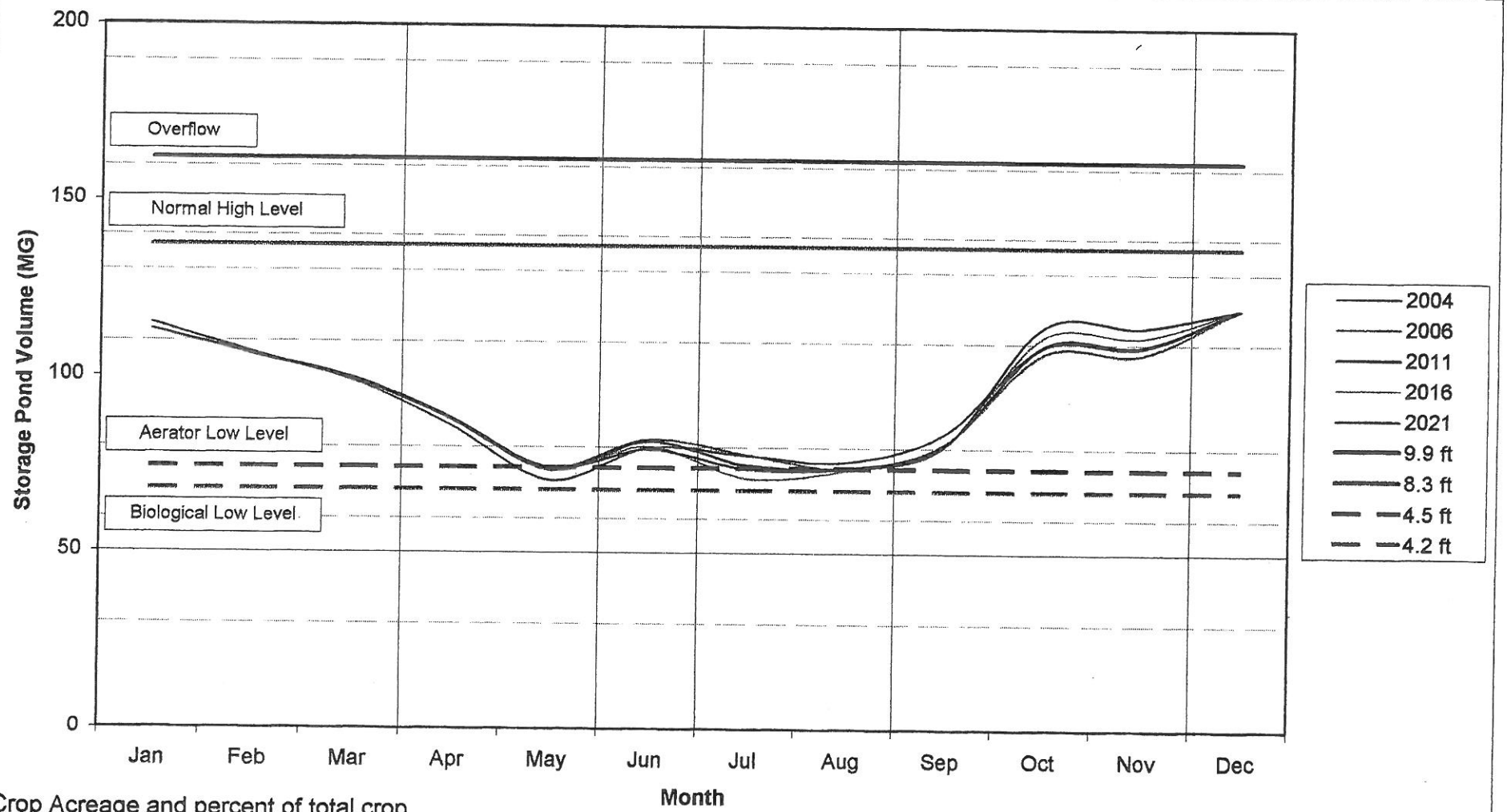
Crop acreage and percent of total crop demand satisfied:
2004: 117 ac trees @ 71%
2006: 367 ac trees @ 70% (incl. 250 ac of 1-yr poplar)
2011: 312 ac trees @ 70%
2016: 372 ac trees @ 70%
2021: 425 ac trees @ 70%, 242 ac grass @ 3%

Figure B-6
Monmouth WWTP
Storage Pond Water Balance
Combined Flow - Design Case #2

Design Case #3

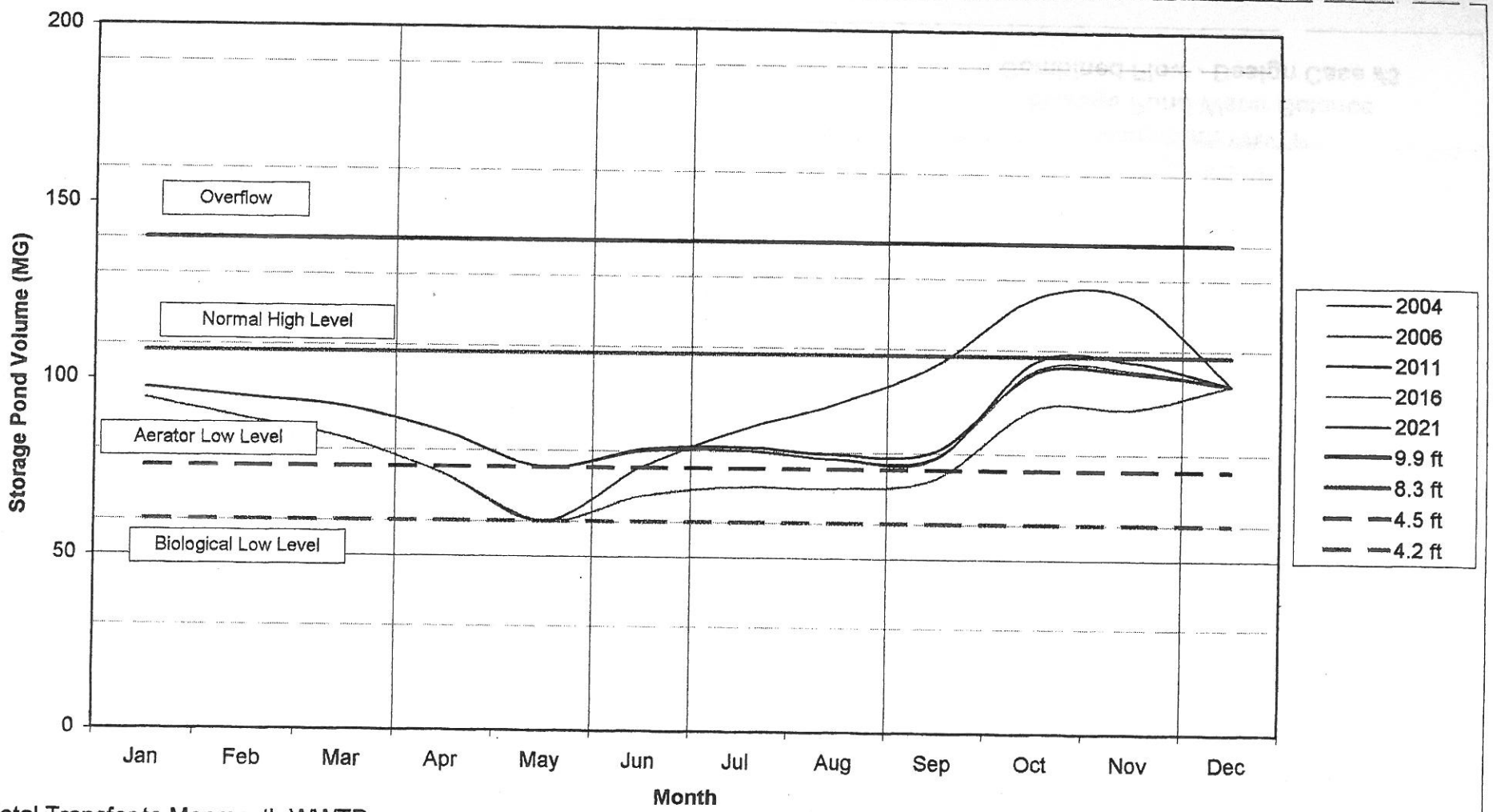
This design case assumes that both communities will grow at a rates that are similar to recent growth trends, precipitation will be at a ten year low, and evaporation will be 107% of average (corresponding with 10-yr low precipitation). The pond volume at the end of May must correspond with that in Design Case #2.

The crop acreages shown in this design case are the same as those shown in Design Case #2. The amount of irrigation has been reduced to maintain the minimum volume in the lagoons. The purpose of this design case is to quantify the crop water deficit that would exist on a year when precipitation was at a 10-year low. This information allows the landowner to determine if on-site storage is required to ensure crop productivity on a dry year.



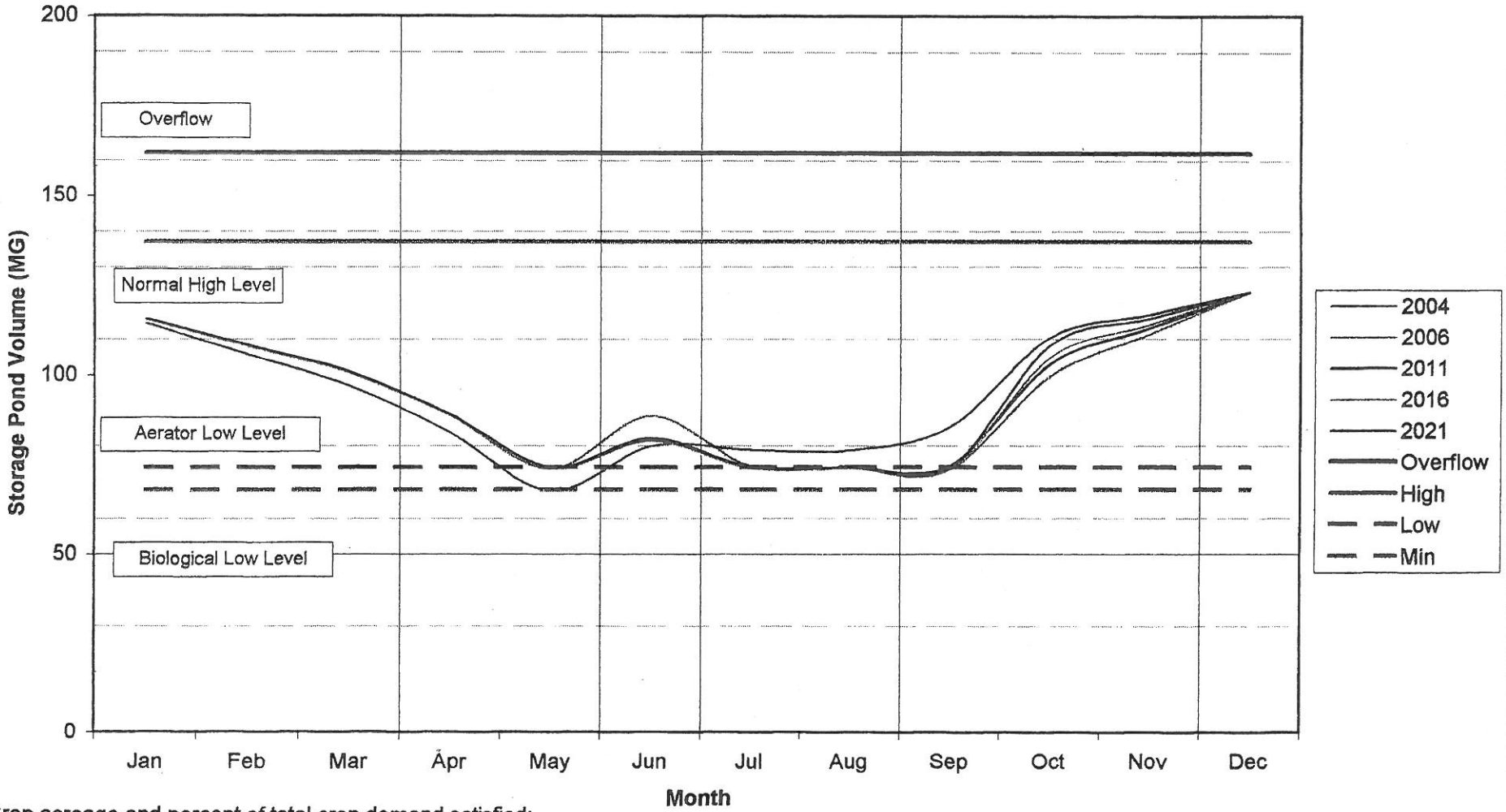
Crop Acreage and percent of total crop demand satisfied:
2004: 117 ac trees @ 73%
2006: 137 ac trees @ 74%
2011: 157 ac trees @ 70%
2016: 185 ac trees @ 66%
2021: 216 ac trees @ 62%

Figure B-7
Monmouth WWTP
Storage Pond Water Balance
Monmouth Flows Only - Design Case #3



Total Transfer to Monmouth WWTP:
2004: 0 MG
2006: 21 MG
2011: 47 MG
2016: 60 MG
2021: 74 MG

Figure B-8
Independence WWTP
Storage Pond Water Balance
Design Case #3



Crop acreage and percent of total crop demand satisfied:
2004: 117 ac trees @ 72%
2006: 367 ac trees @ 69% (incl.250 ac of 1-yr Poplar)
2011: 312 ac trees @ 59%
2016: 372 ac trees @ 57%
2021: 425 ac trees @ 57%, 242 ac grass @ 7%

Figure B-9
Monmouth WWTP
Storage Pond Water Balance
Combined Flow - Design Case #3

City of Independence
WWTP Lagoon Physical Characteristics

Lagoon No.	Bottom El ft	Top of Dike ft	Min FBD ft	Min Depth ft	HGL_{max} ft	HGL_{min} ft	A_{avg} ac	V_{ovrflo} Mgal	V_{max} Mgal	V_{min} Mgal	Available Storage Mgal
1	164.5	174.5	2.0	4.0	172.5	168.5	11.1	36.2	28.9	14.5	14.5
2	171.5	178.3	2.0	4.0	176.3	175.5	9.2	20.3	14.3	11.9	2.4
3	165.5	172.5	2.0	4.0	170.5	169.5	14.2	32.4	23.1	18.5	4.6
4	160.5	171.0	2.0	3.0	169.0	163.5	15.1	51.7	41.8	14.8	27.1
						Total =	49.6	140.5	108.2	59.7	48.5

Min FBD = minimum freeboard, typically 2 ft.
Min Depth = minimum depth, DEQ recommends minimum of 3.5 ft.
HGL_{max} = maximum level in pond.
HGL_{min} = minimum level in pond.
A_{avg} = Average area of pond (at average level).[1]
V_{ovrflo} = Volume of pond at overflow condition.
V_{max} = Volume of pond at maximum level.[1]
V_{min} = Volume of pond at minimum level.[1]

[1] ASCG, *City of Independence Sewage Facilities Plan Update*, 1994, Appendix G

Appendix C

Unit Process Capacity Calculations

Case #1 - No Improvements, Contact Time in Monmouth CCB Alone

Start Date:	15-May		
End Date:	1-Oct		
Irrig. Season Length	139	days	
Monmouth CCB Volume:	74,530	gallons w/CCB at 9 ft depth	
Consider Monmouth Pipeline to STA:			
Monmouth Pipe Diameter:	16	inches	
Pipeline Volume:	0	gallons	
Independence CCB Volume:		gallons	
Consider Independence Pipeline to STA:			
Independence Pipe Diameter:	10	inches	
Pipeline Volume:	0	gallons	
Total Volume:	74,530		
Contact Time @ Avg Condition:	60	min	
Contact Time @ Peak Condition:	20	min	
Capacity for 60 min contact time (avg cond.):	1,242	gpm	
Capacity for 20 min contact time (peak cond.):	3,727	gpm	
Irrigation Pump Capacity:	1,350	gpm	EXISTING
Backwash Rate:	150	gpm	EXISTING
Net Irrigation Pump Capacity:	1,200	gpm	
Reduction for Sustained Flow:	10%		
Effective Irrigation Pump Capacity	1,085		
Strainer Capacity:	1,700		EXISTING
Pipeline Capacity:	2,950	gpm	EXISTING
Irrigation Site Capacity:	1,200	gpm	EXISTING
Max Irrigation Flow at Dry/Avg Conditions:	**1,085**	**gpm**	
Max Irrigation Flow at Peak Conditions:	**1,085**	**gpm**	

Condition	2004			2021		
	Dry	Avg	Peak	Dry	Avg	Peak
Max Flow Rate, gpm	1,085	1,085	1,085	1,085	1,085	1,085
Design Irrigation Days/Week	5	5	5	5	5	7
Min/Req'd Irrig Hrs/d						
Monmouth Only	5	7	7	12	14	11
Combined	9	14	14	23	28	23

Conclusions:
The existing facilities are adequate for current flows from Monmouth but will not accommodate future flows from Monmouth or combined flows from Monmouth/Independence without requiring an irrigation schedule that exceeds established criteria.

Case #1A - No Improvements, Contact Time in Monmouth CCB and Pipe to STA7+00, 2016 Flows

Start Date:	15-May		
End Date:	1-Oct		
Irrig. Season Length	139	days	
Monmouth CCB Volume:	74,530	gallons w/CCB at 9 ft depth	
Consider Monmouth Pipeline to STA:	7+00		
Monmouth Pipe Diameter:	16	inches	
Pipeline Volume:	7,311	gallons	
Independence CCB Volume:		gallons	
Consider Independence Pipeline to STA:			
Independence Pipe Diameter:	10	inches	
Pipeline Volume:	0	gallons	
Total Volume:	81,841		
Contact Time @ Avg Condition:	60	min	
Contact Time @ Peak Condition:	20	min	
Capacity for 60 min contact time (avg cond.):	1,364	gpm	
Capacity for 20 min contact time (peak cond.):	4,092	gpm	
Irrigation Pump Capacity:	1,350	gpm	EXISTING
Backwash Rate:	150	gpm	EXISTING
Net Irrigation Pump Capacity:	1,200	gpm	
Reduction for Sustained Flow:	10%		
Effective Irrigation Pump Capacity	1,085		
Strainer Capacity:	1,700		EXISTING
Pipeline Capacity:	2,950	gpm	EXISTING
Irrigation Site Capacity:	1,200	gpm	EXISTING
Max Irrigation Flow at Dry/Avg Conditions:	**1,085**	**gpm**	
Max Irrigation Flow at Peak Conditions:	**1,085**	**gpm**	

Condition	2004			***2016***		
	Dry	Avg	Peak	Dry	Avg	Peak
Max Flow Rate, gpm	1,085	1,085	1,085	1,085	1,085	1,085
Design Irrigation Days/Week	5	5	5	5	5	7
Min/Req'd Irrig Hrs/d						
Monmouth Only	5	7	7	9	12	9
Combined	9	14	14	23	28	23

Conclusions:
Existing facilities are adequate for current and future flows through 2016 from Monmouth if contact time is considered out to the indicated pipe station in the transmission line. However, the improvements and additional contact time will not accommodate future combined flows from Monmouth/Independence without requiring an irrigation schedule that exceeds established criteria.

Case #2 - With Improvements, Contact Time in Monmouth CCB and Pipe to STA 7+00

Start Date:	15-May		
End Date:	1-Oct		
Irrig. Season Length	139	days	
Monmouth CCB Volume:	74,530	gallons w/CCB at 9 ft depth	
Consider Monmouth Pipeline to STA:	7+00		
Monmouth Pipe Diameter:	16	inches	
Pipeline Volume:	7,311	gallons	
Independence CCB Volume:		gallons	
Consider Independence Pipeline to STA:			
Independence Pipe Diameter:	10	inches	
Pipeline Volume:	0	gallons	
Total Volume:	81,841		
Contact Time @ Avg Condition:	60	min	
Contact Time @ Peak Condition:	20	min	
Capacity for 60 min contact time (avg cond.):	1,364	gpm	
Capacity for 20 min contact time (peak cond.):	4,092	gpm	
Irrigation Pump Capacity:	1,500	gpm	IMPROVED
Backwash Rate:	150	gpm	EXISTING
Net Irrigation Pump Capacity:	1,350	gpm	
Reduction for Sustained Flow:	10%		
Effective Irrigation Pump Capacity	1,221		
Strainer Capacity:	1,700		EXISTING
Pipeline Capacity:	2,950	gpm	EXISTING
Irrigation Site Capacity:	1,350	gpm	IMPROVED
Max Irrigation Flow at Dry/Avg Conditions:	**1,214**	**gpm**	
Max Irrigation Flow at Peak Conditions:	**1,221**	**gpm**	

Condition	2004			2021		
	Dry	Avg	Peak	Dry	Avg	Peak
Max Flow Rate, gpm	1,214	1,214	1,221	1,214	1,214	1,221
Design Irrigation Days/Week	5	5	5	5	5	7
Min/Req'd Irrig Hrs/d						
Monmouth Only	4	6	6	10	12	10
Combined	8	13	12	21	25	20

Conclusions:

Improved facilities are adequate for current and future flows from Monmouth if contact time is considered out to the indicated pipe station in the transmission line. However, the improvements and additional contact time will not accommodate future combined flows from Monmouth/Independence without requiring an irrigation schedule that exceeds established criteria.

Case #3 - With Improvements, Contact Time in CCB and Pipe to STA 70+00

Start Date:	15-May		
End Date:	1-Oct		
Irrig. Season Length	139	days	
Monmouth CCB Volume:	74,530	gallons w/CCB at 9 ft depth	
Consider Monmouth Pipeline to STA:	70+00		
Monmouth Pipe Diameter:	16	inches	
Pipeline Volume:	73,108	gallons	
Independence CCB Volume:		gallons	
Consider Independence Pipeline to STA:			
Independence Pipe Diameter:	10	inches	
Pipeline Volume:	0	gallons	
Total Volume:	147,638		
Contact Time @ Avg Condition:	60	min	
Contact Time @ Peak Condition:	20	min	
Capacity for 60 min contact time (avg cond.):	2,461	gpm	
Capacity for 20 min contact time (peak cond.):	7,382	gpm	
Irrigation Pump Capacity:	2,700	gpm	IMPROVED
Backwash Rate:	300	gpm	IMPROVED
Net Irrigation Pump Capacity:	2,400	gpm	
Reduction for Sustained Flow:	10%		
Effective Irrigation Pump Capacity	2,170		
Strainer Capacity:	3,400		IMPROVED
Pipeline Capacity:	2,950	gpm	EXISTING
Irrigation Site Capacity:	2,400	gpm	IMPROVED
Max Irrigation Flow at Dry/Avg Conditions:	**2,161**	**gpm**	
Max Irrigation Flow at Peak Conditions:	**2,170**	**gpm**	

	2004			2021		
Condition	Dry	Avg	Peak	Dry	Avg	Peak
Max Flow Rate, gpm	2,161	2,161	2,170	2,161	2,161	2,170
Design Irrigation Days/Week	5	5	5	5	5	7
Min/Req'd Irrig Hrs/d						
Monmouth Only	2	3	3	6	7	6
Combined	5	7	7	12	14	11

Conclusions:
Improved facilities are adequate for current and future flows from Monmouth if contact time is considered out to the indicated pipe station in the transmission line. However, chlorine contact time is insufficient for future combined flows from Monmouth/Independence without requiring an irrigation schedule that exceeds established criteria.

Case #4 - With Improvements, Contact Time in CCB and Pipe to STA 90+00

Start Date:	15-May		
End Date:	1-Oct		
Irrig. Season Length	139	days	
Monmouth CCB Volume:	74,530	gallons w/CCB at 9 ft depth	
Consider Monmouth Pipeline to STA:	90+00		
Monmouth Pipe Diameter:	16	inches	
Pipeline Volume:	93,996	gallons	
Independence CCB Volume:		gallons	
Consider Independence Pipeline to STA:			
Independence Pipe Diameter:	10	inches	
Pipeline Volume:	0	gallons	
Total Volume:	168,526		
Contact Time @ Avg Condition:	60	min	
Contact Time @ Peak Condition:	20	min	
Capacity for 60 min contact time (avg cond.):	2,809	gpm	
Capacity for 20 min contact time (peak cond.):	8,426	gpm	
Irrigation Pump Capacity:	3,000	gpm	IMPROVED
Backwash Rate:	300	gpm	IMPROVED
Net Irrigation Pump Capacity:	2,700	gpm	
Reduction for Sustained Flow:	10%		
Effective Irrigation Pump Capacity	2,441		
Strainer Capacity:	3,400		IMPROVED
Pipeline Capacity:	2,950	gpm	EXISTING
Irrigation Site Capacity:	2,700	gpm	IMPROVED
Max Irrigation Flow at Dry/Avg Conditions:	**2,441**	**gpm**	
Max Irrigation Flow at Peak Conditions:	**2,441**	**gpm**	

Condition	2004			2021		
	Dry	Avg	Peak	Dry	Avg	Peak
Max Flow Rate, gpm	2,441	2,441	2,441	2,441	2,441	2,441
Design Irrigation Days/Week	5	5	5	5	5	7
Min/Req'd Irrig Hrs/d						
Monmouth Only	2	3	3	5	6	5
Combined	4	6	6	10	12	10

Conclusions:
Improved facilities are adequate for current and future combined flows from Monmouth and Independence if contact time is considered out to the indicated pipe station in the transmission line.

Table C-1 - Monmouth Irrigation Pump Station and Transmission Pipe System Calculations

Single Pump Operation

Total Flow = 2955 gpm = 6.58 cfs

Description	Qty	Pipe Length	K(new)	Line Size	Flow	Velocity	Minor Losses	Leq @ Smooth C value	Total Length	Dynamic Losses C = 150
Suction Piping										
Entrance	1		1	8	1478	9.4	1.380	47.6	47.6	1.38
Pipe		10		8	1478	9.4	0.000	0.0	10.0	0.29
Discharge Piping										
Bellows Connector	1		1	8	1478	9.4	1.380	47.6	47.6	1.38
Pipe		1		8	1478	9.4	0.000	0.0	1.0	0.03
Swing Check Valve	1		1.4	8	1478	9.4	1.932	66.6	66.6	1.93
Butterfly Valve	1		0.8	8	1478	9.4	1.104	38.1	38.1	1.10
90 degree elbow	1		0.3	8	1478	9.4	0.414	14.3	14.3	0.41
Pipe		2		8	1478	9.4	0.000	0.0	2.0	0.06
Reducer	0		0.25	8	1478	9.4	0.345	11.9	0.0	0.00
Branch Flow Tee	1		0.84	8	1478	9.4	1.159	40.0	40.0	1.16
45 degree elbow	1		0.13	8	1478	9.4	0.179	6.2	6.2	0.18
Butterfly Valve	1		0.8	8	1478	9.4	1.104	38.1	38.1	1.10
Clean Strainer	1		7	10	1478	6.0	3.957	403.9	403.9	3.96
Butterfly Valve	1		0.8	8	1478	9.4	1.104	38.1	38.1	1.10
45 degree elbow	1		0.13	8	1478	9.4	0.179	6.2	6.2	0.18
Branch Flow Tee	1		0.84	8	1478	9.4	1.159	40.0	40.0	1.16
Pipe		2		8	1478	9.4	0.000	0.0	2.0	0.06
90 degree elbow	1		0.3	8	1478	9.4	0.414	14.3	14.3	0.41
Wafer Check Valve	1		0.8	8	1478	9.4	1.104	38.1	38.1	1.10
Pipe		2		8	1478	9.4	0.000	0.0	2.0	0.06
Reducer	1		0.13	8	1478	9.4	0.179	6.2	6.2	0.18
Thru Flow Tee	1		0.28	10	1478	6.0	0.158	16.2	16.2	0.16
Swing Check Valve	1		1.4	10	2955	12.1	3.165	89.6	89.6	3.17
Pipe		1		10	2955	12.1	0.000	0.0	1.0	0.04
Thru Flow Tee	1		0.28	10	2955	12.1	0.633	17.9	17.9	0.63
Pipe		1		10	2955	12.1	0.000	0.0	1.0	0.04
90 degree elbow	1		0.25	10	2955	12.1	0.565	16.0	16.0	0.57
Pipe		3		10	2955	12.1	0.000	0.0	3.0	0.11
Reducer	1		0.13	10	2955	12.1	0.294	8.3	8.3	0.29
90 degree elbow	1		0.25	16	2955	4.7	0.086	24.0	24.0	0.09
PVC Trans. Pipe		17500		16	2955	4.7	0.000	0.0	17500.0	62.78
Misc Losses	1		25	16	2955	4.7	8.625	2404.0	2404.0	8.62

Suction Calculations

Wet Well Level =	165	Ha =	33.96
Pump Centerline El =	172	Hvpa =	0.783
Hst =	-7		

Discharge El = 244.00
Dischrg Pressure = 25 psig

Total Dynamic Loss = 94
Total Static Head = 136.6
Total Dynamic Head = 230
NPSHa = 24.5

Table C-2 - Independence Effluent Transfer Pump Station System Calculations

Single Pump Operation

Total Flow = 750 gpm = 1.67 cfs

Description	Qty	Pipe Length	K(new)	Line Size	Flow	Velocity	Minor Losses	Leq @ Smooth C value	Total Length	Dynamic Losses C = 150
Suction Piping										
Entrance	1		0.78	10	750	3.1	0.114	40.7	40.7	0.11
Discharge Piping										
90 degree elbow	1		0.28	10	750	3.1	0.041	14.6	14.6	0.04
Pipe		15		10	750	3.1	0.000	0.0	15.0	0.04
90 degree elbow	1		0.28	10	750	3.1	0.041	14.6	14.6	0.04
Pipe		2		10	750	3.1	0.000	0.0	2.0	0.01
Swing Check Valve	1		1.4	10	750	3.1	0.204	73.0	73.0	0.20
Plug Valve	1		0.25	10	750	3.1	0.036	13.0	13.0	0.04
Through Flow Tee	1		0.28	10	750	3.1	0.041	14.6	14.6	0.04
Plug Valve	1		0.25	10	750	3.1	0.036	13.0	13.0	0.04
Pipe		8200		10	750	3.1	0.000	0.0	8200.0	22.91
Exit Loss	1		1	10	750	3.1	0.146	52.1	52.1	0.15

Suction Calculations

Wet Well Level =	160	Ha =	33.96
Pump Centerline El =	162	Hvpa =	0.783
Hst =	-2		

Discharge El = 180.00

Dischrg Pressure = 0 psig

Total Dynamic Loss = 23.62

Total Static Head = 20.0

Total Dynamic Head = 44

NPSHa = 31.1

Appendix D

Economic Analysis

Table D-1 - Economic Rate Analysis

Year	Cost	Ind. Qty	Indep. Rate	Ind. Income	Independence Services	Avg. Cost/Svc/Mo.	Comb. Qty	Demand	Add'l Ag Qty	Ag Rate	Ag. Income
2000	-$340,000			$0							$0
2001	-$2,852,000			$0							$0
2002	-$64,232			$0							$0
2003	-$22,625			$0							$0
2004	-$32,114			$0							$0
2005	-$30,250			$0			61	61	0		$0
2006	-$44,256	50	$3,270	$163,824	2,800	$4.9	64	100	36	$200	$7,224
2007	-$46,332	52	$3,270	$170,213	2,877	$4.9	67	120	53	$200	$10,618
2008	-$48,516	54	$3,270	$176,852	2,956	$5.0	70	140	70	$200	$13,983
2009	-$50,815	56	$3,270	$183,749	3,037	$5.0	73	160	87	$200	$17,315
2010	-$53,234	58	$3,270	$190,915	3,120	$5.1	77	175	98	$200	$19,614
2011	-$55,782	61	$3,270	$198,361	3,205	$5.2	81	175	94	$200	$18,878
2012	-$58,164	63	$3,270	$206,097	3,293	$5.2	84	175	91	$250	$22,631
2013	-$60,673	65	$3,270	$214,135	3,383	$5.3	89	175	86	$250	$21,615
2014	-$63,302	68	$3,270	$222,486	3,476	$5.3	93	175	82	$250	$20,547
2015	-$66,057	71	$3,270	$231,163	3,571	$5.4	97	175	78	$250	$19,426
2016	-$68,946	73	$3,270	$240,178	3,669	$5.5	102	175	73	$250	$18,246
2017	-$71,976	76	$3,270	$249,545	3,770	$5.5	107	175	68	$300	$20,408
2018	-$75,154	79	$3,270	$259,277	3,873	$5.6	112	175	63	$300	$18,845
2019	-$78,488	82	$3,270	$269,389	3,979	$5.6	118	175	57	$300	$17,201
2020	-$81,987	86	$3,270	$279,895	4,088	$5.7	123	175	52	$300	$15,472
2021	-$85,659	89	$3,270	$290,811	4,200	$5.8	129	175	46	$300	$13,654
							129	175	46	$300	$13,654

Discount Rate	7%	
2000 PW Costs =	-$3,530,000	
2000 PW Indep. Income =	$1,410,000	
2000 PW Ag. Income =	$0	Assume zero, unproven source of income

2000 PW Ag. Income = $120,000

		Share
Monmouth Net Prsnt Value (Cost) =	-$2,120,000	52%
Indep. Total 2000 PW Cost =	-$1,960,000	48%

Table D-2 - Estimated Annual Costs to City of Monmouth Associated with Hydraulic Capacity Related Improvements of the Effluent Reclamation Facilities as a Result of Monmouth Growth Alone

	Monmouth Costs - Monmouth Effluent					
	Power	Labor	Engr	Capital	Total	
2000	$0	$40,000	$300,000		$340,000	Permitting & Design
2001	$2,000	$50,000	$300,000	$2,500,000	$2,852,000	Ph I Design & Const.
2002	$3,000	$11,232	$50,000		$64,232	O&M and Engr. Support
2003	$4,000	$11,625	$7,000		$22,625	O&M and Engr. Support
2004	$4,366	$12,032	$10,000		$26,398	O&M and Engr. Support
2005	$4,677	$12,453	$7,000		$24,131	O&M and Engr. Support
2006	$5,012	$12,889	$7,350		$25,251	O&M and Engr. Support
2007	$5,371	$13,340	$7,718		$26,428	O&M and Engr. Support
2008	$5,756	$13,807	$8,103		$27,666	O&M and Engr. Support
2009	$6,169	$14,290	$8,509		$28,968	O&M and Engr. Support
2010	$6,612	$14,790	$8,934		$30,336	O&M and Engr. Support
2011	$7,088	$15,308	$9,381		$31,776	O&M and Engr. Support
2012	$7,598	$15,844	$9,850		$33,292	O&M and Engr. Support
2013	$8,146	$16,398	$10,342		$34,886	O&M and Engr. Support
2014	$8,733	$16,972	$10,859		$36,565	O&M and Engr. Support
2015	$9,364	$17,566	$11,402		$38,332	O&M and Engr. Support
2016	$10,040	$18,181	$11,972	$75,000	$115,194	Ph. II Imp., Engr Support, O&M
2017	$10,766	$18,818	$12,571		$42,155	O&M and Engr. Support
2018	$11,546	$19,476	$13,200		$44,221	O&M and Engr. Support
2019	$12,382	$20,158	$13,860		$46,399	O&M and Engr. Support
2020	$13,279	$20,863	$14,552		$48,695	O&M and Engr. Support
2021	$14,243	$21,594	$15,280		$51,116	O&M and Engr. Support

Discount Rate	7%
2000 PW Costs =	$3,370,000

Table D-3 - Estimated Annual Costs to City of Monmouth Associated with Hydraulic Capacity Related Improvements of the Effluent Reclamation Facilities as a Result of Combined Growth of Independence and Monmouth

	Monmouth Costs - Combined Effluent					
	Power	Labor	Engr	Capital	Total	
2000	$0	$40,000	$300,000		$340,000	Permitting & Design
2001	$2,000	$50,000	$300,000	$2,500,000	$2,852,000	Ph I Design & Const.
2002	$3,000	$11,232	$50,000		$64,232	O&M and Engr. Support
2003	$4,000	$11,625	$7,000		$22,625	O&M and Engr. Support
2004	$10,082	$12,032	$10,000		$32,114	O&M and Engr. Support
2005	$10,797	$12,453	$7,000		$30,250	O&M and Engr. Support
2006	$11,564	$25,342	$7,350		$44,256	O&M and Engr. Support
2007	$12,385	$26,229	$7,718		$46,332	O&M and Engr. Support
2008	$13,266	$27,147	$8,103		$48,516	O&M and Engr. Support
2009	$14,209	$28,097	$8,509		$50,815	O&M and Engr. Support
2010	$15,220	$29,081	$8,934		$53,234	O&M and Engr. Support
2011	$16,303	$30,098	$9,381		$55,782	O&M and Engr. Support
2012	$17,162	$31,152	$9,850		$58,164	O&M and Engr. Support
2013	$18,088	$32,242	$10,342		$60,673	O&M and Engr. Support
2014	$19,072	$33,371	$10,859		$63,302	O&M and Engr. Support
2015	$20,116	$34,539	$11,402		$66,057	O&M and Engr. Support
2016	$21,226	$35,748	$11,972		$68,946	O&M and Engr. Support
2017	$22,406	$36,999	$12,571		$71,976	O&M and Engr. Support
2018	$23,660	$38,294	$13,200		$75,154	O&M and Engr. Support
2019	$24,994	$39,634	$13,860		$78,488	O&M and Engr. Support
2020	$26,413	$41,021	$14,552		$81,987	O&M and Engr. Support
2021	$27,922	$42,457	$15,280		$85,659	O&M and Engr. Support

Discount Rate 7%
2000 PW Costs = $3,530,000

Table D-4 - Estimated Annual Costs to City of Independence Associated with Development of a Combined Effluent Reclamation Project with City of Monmouth

	Independence Costs - Combined Effluent					
	Rate Total	Labor	Engr	Capital	Total	
2000	$0				$0	Permitting & Design
2001	$0				$0	Ph I Design & Const.
2002	$0				$0	O&M and Engr. Support
2003	$0				$0	O&M and Engr. Support
2004	$0				$0	O&M and Engr. Support
2005	$0				$0	O&M and Engr. Support
50m gal — 2006	$163,824	$30,000	$120,000	$600,000	$913,824	Ph. II Imp., Engr Support, O&M
2007	$170,213	$5,616			$175,829	O&M and Engr. Support
2008	$176,852	$5,813			$182,664	O&M and Engr. Support
2009	$183,749	$6,016			$189,765	O&M and Engr. Support
2010	$190,915	$6,227			$197,142	O&M and Engr. Support
2011	$198,361	$6,444			$204,805	O&M and Engr. Support
2012	$206,097	$6,670			$212,767	O&M and Engr. Support
2013	$214,135	$6,903			$221,038	O&M and Engr. Support
2014	$222,486	$7,145			$229,631	O&M and Engr. Support
2015	$231,163	$7,395			$238,558	O&M and Engr. Support
2016	$240,178	$7,654			$247,832	O&M and Engr. Support
2017	$249,545	$7,922			$257,467	O&M and Engr. Support
2018	$259,277	$8,199			$267,476	O&M and Engr. Support
2019	$269,389	$8,486			$277,875	O&M and Engr. Support
2020	$279,895	$8,783			$288,678	O&M and Engr. Support
89m gal — 2021	$290,811	$9,091			$299,902	O&M and Engr. Support

Discount Rate	7%
2000 PW Costs =	-$1,960,000

08 February 2013

Technical Memorandum - DRAFT

To: Mr. Russell Cooper, City of Monmouth

From: Monty Hazlehurst, Kennedy/Jenks Consultants

Reviewed: Ron Walz, Kennedy/Jenks Consultants

Subject: City of Monmouth - City of Independence Joint Effluent Reuse Facility
K/J 1076001*00

The purpose of this Technical Memorandum (Tech Memo) is to develop the City of Monmouth effluent reuse facility improvements which would be necessary for the City of Monmouth Wastewater Treatment Facility (WWTF) to receive treated effluent from the City of Independence WWTF during the dry season, combine it with the City of Monmouth WWTF effluent, and pump it to land application sites. This Tech Memo presents the costs estimates associated with the improvements, including the City of Independence's share of the costs.

The estimated costs include existing effluent reuse facility costs, future effluent reuse facility costs, and annual operations and maintenance costs associated with the effluent reuse facilities. The analysis is based on the assumption that the City of Independence's flows will be delivered to the City of Monmouth WWTF in the year 2014 and the planning period runs to the year 2035.

Summary

In order for the two Cities to develop combined effluent reuse facilities, they will need to develop an intergovernmental agreement which includes the "buy-in" cost for the City of Independence and a guarantee from the City of Independence that they will deliver the estimated quantity of effluent through the planning period and that they will be responsible for their portion of the annual O&M costs.

The estimated City of Independence "buy in" costs associated with the City of Monmouth effluent reuse facilities are as follows:

Existing Effluent Reuse Facilities: $1,312,000
Present Worth of Initial Improvements to Effluent Reuse Pump Station: $63,000
Present Worth of Effluent Reuse Plan Update: $51,000
Present Worth of Future Effluent Reuse Facilities: $397,000
Total Cost: $1,823,000

The estimated average O&M cost over the planning period for the City of Independence is $27,000 per year and represents an estimated average rate of $388 per million gallons (Mgal) of effluent from the City of Independence.

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Assumptions

It is assumed the City of Independence will provide the following in a combined system:

- Pumping facilities at the City of Independence WWTF, including pump controls based on water levels in the City of Monmouth WWTF reuse pump station wet well.
- Transmission pipeline from the City of Independence WWTF to the City of Monmouth WWTF reuse pump station wet well.
- Magnetic flow meter and shut off valve at the end of the transmission pipeline at the City of Monmouth WWTF reuse pump station.
- Delivery of disinfected effluent to the City of Monmouth WWTF reuse pump station. Delivered effluent shall meet all regulatory requirements, including all future regulatory requirements, as defined in the National Pollution Discharge Elimination System (NPDES) permit in effect.

All costs for these items (including design costs, capital costs, and annual operation and maintenance costs) would be paid for by the City of Independence and are beyond the scope of this Tech Memo.

Background

In 2004 a Monmouth/Independence Joint Effluent Reclamation Project Feasibility Study was developed by Whitaker Engineering. Major differences between the data presented in the Feasibility Study and this Tech Memo are as follows:

- The Feasibility Study assumed there would be no storage of effluent at the City of Independence WWTF during the dry weather period. This Tech Memo presents data assuming the City of Independence will continue to store effluent during the dry weather period equivalent to what is currently stored.
- The Feasibility Study developed total project costs (including capital and annual O&M costs) for the City of Monmouth and the City of Independence, including costs for the City of Independence WWTF reuse pump station and transmission pipeline to the City of Monmouth WWTF. As stated above, the analysis presented in this Tech Memo does not include those costs.
- The Feasibility Study developed estimated annual costs for the City of Independence based on the cost of existing and future effluent reuse facilities for both Monmouth and

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Independence combined with annual O&M costs for both Monmouth and Independence and did not include a "buy-in" cost. The analysis presented in this Tech Memo includes an estimated City of Independence "buy-in" cost for the existing and future effluent reuse facilities at the City of Monmouth and the estimated City of Independence portion of the annual O&M cost for the effluent reuse facilities at the City of Monmouth.

Dry Weather Effluent Flows

Effluent from the two facilities cannot be discharged to receiving waters from June 1st through October 31st (153 calendar days) as a permit condition. The City of Independence WWTF currently stores effluent within its treatment lagoons during this period, estimated at 57 Mgal over the 153 day period. Assuming that the City of Independence will continue to store 57 Mgal in its lagoons each year in the future, any additional flows will need to be pumped to reuse facilities.

The attached Table 1 presents the estimated projected flows from each facility and the combined flows. The City of Independence effluent flow represents an increasing percent of the combined flow, reaching 38.4 percent in 2035.

During the first half of June and the last half of October it is often times too rainy to irrigate. This analysis assumes that during this period the Cities of Independence and Monmouth will have to store their effluent and that the dry weather pumping period will be reduced from 153 days to 123 days. Therefore, the two facilities will need to pump the flow generated in 153 days over a 123 day period.

Existing Effluent Reuse Facilities

In the years 2000 through 2001, the City of Monmouth designed and constructed the Phase 1 effluent reuse and conveyance facilities including an effluent reuse pump station, an effluent transmission pipeline, and an irrigation site booster pump station. Assuming a maximum pressure of 100 pounds per square inch gauge (psig) at the reuse pump station, the transmission pipeline has a capacity of 2,955 gallons per minute (gpm).

In the years 2010 through 2013, the City of Monmouth designed and constructed the Phase 2 effluent reuse facilities which include the addition of a second pump and strainer at the effluent reuse pump station. After the Phase 2 facilities are complete, the effluent reuse pump station will have a firm capacity (capacity of a pump station with the largest pump out of service) of 1,350 gpm at 166 feet total dynamic head (TDH). By designing a pump station to have a firm capacity equal to or greater than the required pumping rate, it allows the required pumping rate to be achieved even if the largest pump breaks down. The reuse pumps are 75 horsepower (hp) each.

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The costs of these existing effluent reuse facilities are presented in the attached Table 2. The total cost of the existing effluent reuse facilities is $3,416,000. Because the City of Independence's share of the combined flow at the design year 2035 is estimated at 38.4 percent, that percentage should also be applied to the cost of the existing facilities in order to estimate the City of Independence's cost for a combined system. The City of Independence's share would be $1,312,000.

Future Effluent Reuse Facilities

Future effluent reuse facilities will include initial improvements to receive the City of Independence effluent, developing an effluent reuse plan update, and Phase 3 effluent reuse facility preliminary design, detailed design, and construction.

Initial Improvements. In order for the City of Monmouth WWTF reuse pump station wet well to receive fully disinfected effluent from the City of Independence and continue to properly disinfect the City Monmouth WWTF effluent, it will be necessary to hydraulically separate the effluent reuse pump station wet well from the chlorine contact basin. This could be achieved by placing a wall in the chlorine contact basin just upstream of the effluent reuse pump station wet well. The wall would have a sharp crested weir at the top located at an elevation to maintain the required water depth in the chlorine contact basin, while allowing the water surface elevation in the reuse pump station wet well to vary. The weir wall could be preformed, lowered into the tank with a crane, and bolted into place. This will minimize the time the chlorine contact basin is taken out of service. The reuse pump station controls would be modified to maintain the water level in the wet well.

Effluent Reuse Plan Update. Prior to designing and constructing the future Phase 3 effluent reuse facilities, there will be costs associated with updating the Effluent Reuse Plan with the addition of the Independence flows.

Phase 3 Effluent Reuse Facilities. The projected combined average effluent flows for the year 2035 are estimated to be 1,485 gpm. Assuming the effluent is land applied 12 hours per day, 7 days a week, the reuse pump station would need a capacity of 2,870 gpm. The existing transmission pipeline from the City of Monmouth WWTF to the irrigation sites has a capacity of 2,955 gpm when pumping at 100 psi (230 feet TDH). Therefore, at some point in the future, the two 1,350 gpm, 166 feet TDH, 75 hp pumps, would need to be replaced with three 1,485 gpm, 230 TDH, 125 hp pumps to provide a firm capacity of 2,870 gpm.

Assuming a maximum irrigation interval of 12 hours per day, 7 days a week, the existing pump station firm capacity of 1,350 gpm will meet the system requirements up to a combined effluent flow of 675 gpm. It is estimated the combined system would reach this flow in year 2017. Therefore, the reuse pump station improvements would need to be constructed by year 2017.

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During Phase 3, the irrigation site facilities will also need to be expanded including upgrading the booster pump station and additional irrigation equipment.

Estimated Future Facility Costs. The attached Table 3 presents the estimated costs of the future effluent reuse facility improvements. The reuse pump station improvements in year 2014, with an estimated present worth of $63,000, would be paid for by the City of Independence. The Effluent Reuse Plan Update with the addition of the Independence flows has an estimated present worth of $51,000, and would be paid for by the City of Independence. The total present worth cost of the year 2016 thru 2017 facilities is $1,035,000. Based on the City of Independence paying 38.4 percent of the future facility costs (based on the design year 2035 flow), the City of Independence's share would be a present worth of $397,000.

Annual Operation and Maintenance Costs

Annual operation and maintenance costs for the effluent reuse facilities were developed based on power and labor for the City of Monmouth to operate the combined effluent reuse facilities. Power costs assume a rate of 8.4 cents/kilowatt hour in 2014 and increasing 2.78 percent per year. Similar values were used in the 2004 Feasibility Study. Labor costs were projected based on labor per Mgal of combined effluent shown in the 2004 Feasibility Study. The attached Table 4 includes the estimated annual O&M costs through year 2035. Table 4 also shows the estimated effluent reuse pump run time in 2035 of 15 hours per day, 7 days per week. In order to reduce the pump run time, the pump station capacity would need to be increased and may require an additional transmission pipeline.

The attached Table 5 summarizes the estimated annual O&M costs for the City of Independence based on the estimated percent of combined flow from the City of Independence of 18 percent in 2014 and rising to 38.4 percent in 2035. The average cost is $27,000 per year and the average rate is $388 per Mgal.

Conclusion

In order for the two Cities to develop combined effluent reuse facilities, they will need to develop an intergovernmental agreement which includes the "buy-in" cost for the City of Independence and a guarantee from the City of Independence that they will deliver the estimated quantity of effluent through the planning period and that they will be responsible for their portion of the annual O&M costs.

The estimated City of Independence "buy in" costs associated with the City of Monmouth effluent reuse facilities are as follows:

Existing Effluent Reuse Facilities: $1,312,000
Present Worth of Initial Improvements to Effluent Reuse Pump Station: $63,000

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Present Worth to Update Effluent Reuse Plan with Independence Flows: $51,000
Present Worth of Future Effluent Reuse Facilities: $397,000
Total Cost: $1,823,000

The estimated average O&M cost over the planning period for the City of Independence is $27,000 per year and represents an estimated average rate of $388 per Mgal of effluent from the City of Independence.

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Table 1. Projected June through October Effluent from Monmouth and Independence

	Monmouth			Independence						Combined			
Year	ADWF[a] mgd	ADW Losses[b] mgd	DW Effluent[c] Mgal	ADWF[a] mgd	ADW Losses[b] mgd	DW Effluent[c] Mgal	Effluent Storage Capacity Mgal	Pumped[d] Effluent Mgal	Pumped[e] Effluent mgd	DW Effluent[c, e] Mgal	DW Effluent[c, e] mgd	DW Effluent[c, e] gpm	Indep. Flow Percent
2014	0.92	0.373	84	0.68	0.182	75	57	18	23	103	0.84	581	18.0
2015	0.95	0.373	89	0.70	0.182	79	57	22	27	111	0.90	624	19.9
2016	0.98	0.373	93	0.72	0.182	83	57	26	32	119	0.96	669	21.7
2017	1.01	0.373	97	0.75	0.182	87	57	30	37	127	1.03	716	23.3
2018	1.04	0.373	102	0.77	0.182	91	57	34	42	135	1.10	764	24.8
2019	1.07	0.373	106	0.80	0.182	95	57	38	47	144	1.17	813	26.1
2020	1.10	0.373	111	0.83	0.182	99	57	42	52	153	1.24	864	27.4
2021	1.13	0.373	116	0.86	0.182	103	57	46	58	162	1.32	917	28.5
2022	1.17	0.373	121	0.89	0.182	108	57	51	63	172	1.40	971	29.5
2023	1.20	0.373	127	0.92	0.182	112	57	55	69	182	1.48	1,028	30.5
2024	1.24	0.373	132	0.95	0.182	117	57	60	75	192	1.56	1,085	31.4
2025	1.27	0.373	137	0.98	0.182	122	57	65	81	203	1.65	1,145	32.2
2026	1.31	0.373	143	1.02	0.182	128	57	71	88	214	1.74	1,207	33.0
2027	1.35	0.373	149	1.05	0.182	133	57	76	94	225	1.83	1,270	33.8
2028	1.39	0.373	155	1.09	0.182	139	57	82	101	237	1.92	1,336	34.4
2029	1.43	0.373	161	1.13	0.182	144	57	87	109	249	2.02	1,404	35.1
2030	1.47	0.373	168	1.16	0.182	150	57	93	116	261	2.12	1,474	35.7
2031	1.51	0.373	174	1.20	0.182	156	57	99	124	274	2.23	1,546	36.3
2032	1.56	0.373	181	1.25	0.182	163	57	106	132	287	2.33	1,620	36.9
2033	1.60	0.373	188	1.29	0.182	169	57	112	140	301	2.44	1,697	37.4
2034	1.65	0.373	195	1.33	0.182	176	57	119	148	315	2.56	1,776	37.9
2035	1.70	0.373	203	1.38	0.182	183	57	126	157	329	2.68	1,858	38.4

Notes:

(a) Average Dry Weather Flow (ADWF) projections are based on projections shown in current Monmouth and Independence Facilities Plans.

(b) Average Dry Weather (ADW) losses equal average daily rainfall minus evaporation over the dry weather period. Because Monmouth has lagoon aerators, it will have higher evaporation losses than Independence (assumes losses equal 2 times non-aerated lagoon losses)

(c) Dry Weather (DW) Period (June through October) equals 153 days

(d) The City of Indepence estimated 55 Mgal would be pumped in 2023 and 90 Mgal would be pumped in 2033. Based on the projected 2033 ADWF, the 2033 pumped flow is 112 Mgal.

(e) Pumped effluent and combined effluent when shown in mgd and gpm assumes a 123 day irrigation period.

Table 2. Estimated Existing City of Monmouth Effluent Reuse Facilitity Costs

Year	Estimated Costs (Dollars)				Item
	Admin[(a)]	Engr[(b)]	Capital	Total	
2000 - 2001	150,000	500,000	2,500,000	3,150,000	Phase 1 Effluent Reuse Facility Design and Construction
2010 - 2013	12,648	42,160	210,800	265,608	Phase 2 Effluent Reuse Facility Expansion Design and Construction
			Total Cost =	$ 3,415,608	
			Independence Cost (38.4%)=	$ 1,311,593	

Notes:

(a) Administration equals 5 percent of Engineering and Capital Costs.

(b) Engineering equals 20 percent of Capital Cost (Design = 10 percent, Construction Management = 10 percent).

Table 3. Estimated Future City of Monmouth Effluent Reuse Facility Costs

Year	Estimated Costs (Dollars)				Item
	Admin[a]	Engr[b]	Capital	Total	
2014	3,000	10,000	50,000	63,000	Weir Wall in Chlorine Contact Basin and Reuse Pump Station Controls
2015	2,500	50,000		52,500	Effluent Reuse Plan Update with Independence Flows
2016	2,500	50,000		52,500	Phase 3 Preliminary Design
2017	50,760	169,200	846,000	1,065,960	Phase 3 Effluent Reuse Facility Expansion Design and Construction

Discount Rate [c] 0.5% in 2014 to 2% in 2017

2013 PW of Total Cost of 2014 Improvements =	$	62,969	(100% Independence Cost)
2013 PW of Total Cost of 2015 Improvements =	$	51,466	(100% Independence Cost)
2013 PW of 2016 thru 2017 Total Cost =	$	1,034,940	
Independence Cost of 2016 thru 2017 Costs (38.4%) =	$	397,417	

Notes:

(a) Administration equals 5 percent of Engineering and Capital Costs.

(b) Engineering equals 20 percent of Capital Cost (Design = 10 percent, Construction Management = 10 percent).

(c) To calculate the present worth, the future cost is "discounted" to the present using the discount rate, by an amount equal to the time value of money. The Federal Office of Management and Budget recommended in 2012 using a discount rate of 1.6 percent for a 3 year time frame.

Table 4. Estimated Annual O&M Costs for City of Monmouth Effluent Reuse Facilities
(Assumes Independence Flows Pumped to Monmouth Starting in Year 2014)

Year	Combined Flow (Mgal)	Pump Run[a] Time (hrs)	Pump Run[b] Time (hrs/day)	Power[a] (kWhrs)	Power Rate[c] (Dollars)	Estimated Annual O&M Costs (Dollars)		
						Power	Labor[d]	Total
2014	102	1259	10	122139	0.08	10,333	25,342	35,675
2015	108	1331	11	129126	0.09	11,228	26,685	37,913
2016	114	1407	11	136512	0.09	12,200	28,099	40,299
2017	121	1488	12	144320	0.09	13,256	29,589	42,845
2018	127	1430	6	180337	0.09	17,025	31,157	48,182
2019	135	1512	6	190653	0.10	18,499	32,808	51,308
2020	142	1598	6	201558	0.10	20,101	34,547	54,648
2021	151	1690	7	213087	0.10	21,842	36,378	58,220
2022	159	1786	7	225276	0.11	23,733	38,306	62,039
2023	168	1889	8	238161	0.11	25,788	40,336	66,124
2024	178	1997	8	251784	0.11	28,021	42,474	70,495
2025	188	2111	9	266186	0.11	30,447	44,725	75,173
2026	199	2232	9	281412	0.12	33,084	47,096	80,180
2027	210	2359	10	297509	0.12	35,949	49,592	85,540
2028	222	2494	10	314526	0.12	39,061	52,220	91,282
2029	235	2637	11	332517	0.13	42,444	54,988	97,432
2030	248	2788	11	351537	0.13	46,119	57,902	104,021
2031	263	2947	12	371645	0.13	50,112	60,971	111,083
2032	278	3116	13	392903	0.14	54,452	64,202	118,654
2033	293	3294	13	415377	0.14	59,167	67,605	126,772
2034	310	3482	14	439137	0.15	64,290	71,188	135,478
2035	328	3681	15	464256	0.15	69,857	74,961	144,818

Notes:

(a) Assumes at Year 2014 the two Reuse Pumps are 1,350 gpm, 166 TDH, 75 hp each.
Assumes at Year 2018 there will be three Reuse Pumps at 1,485 gpm, 230 TDH, 125 hp each.
Assumes Booster Pump run time is equal to Reuse Pump run time. Booster Pumps are 50 hp each.

(b) Pump Run Time (hrs/day) assumes a 123 day irrigation period, irrigating 7 days a week, 1 pump running to 2018, and 2 running 2018-2035.

(c) Assumes power rate inreases by 2.78 percent per year.

(d) Labor was projected based on labor cost/Mgal shown in the 2004 Joint Effluent Feasibility Study.

Table 5. Estimated City of Independence Rates for Independence Portion of Annual O&M Costs.
(Assumes Independence Flows Pumped to Monmouth Starting in Year 2014)

Year	Total Annual O&M Cost (Dollars)	Independence Percent Flow	Independence Annual Cost (Dollars)	Quantity (Mgal)	Rate/Mgal (Dollars)	Number of Services(a)	Cost/Svc/ Month (Dollars)
2014	35,675	18.0	6409	18	356	3,476	0.15
2015	37,913	19.9	7561	20	383	3,571	0.18
2016	40,299	21.7	8753	22	404	3,669	0.20
2017	42,845	23.3	9996	24	421	3,770	0.22
2018	48,182	24.8	11946	26	459	3,873	0.26
2019	51,308	26.1	13406	29	469	3,979	0.28
2020	54,648	27.4	14948	31	477	4,088	0.30
2021	58,220	28.5	16582	34	483	4,200	0.33
2022	62,039	29.5	18317	38	486	4,315	0.35
2023	66,124	30.5	20163	41	488	4,433	0.38
2024	70,495	31.4	22130	45	488	4,555	0.40
2025	75,173	32.2	24230	50	488	4,680	0.43
2026	80,180	33.0	26474	54	486	4,808	0.46
2027	85,768	33.8	28952	60	484	4,940	0.49
2028	91,282	34.4	31446	66	480	5,075	0.52
2029	97,432	35.1	34201	72	476	5,214	0.55
2030	104,021	35.7	37156	79	471	5,357	0.58
2031	111,083	36.3	40328	86	466	5,504	0.61
2032	118,654	36.9	43734	95	461	5,654	0.64
2033	126,772	37.4	47394	104	456	5,809	0.68
2034	135,478	37.9	51328	114	450	5,969	0.72
2035	144,818	38.4	55559	125	444	6,132	0.76
		Average:	27191		388		

Note:
(a) Number of City of Independence services based on projections shown in 2004 Joint Effluent Feasiblility Study.

MEMORANDUM

DATE:	March 22, 2013
TO:	Mike Danko
FROM:	Don Whitehead, P.E.
SUBJECT:	**Review of Joint Effluent Reuse Memorandum**
PROJECT:	SANITARY SEWERAGE SYSTEM FACILITIES PLAN
PROJECT NO:	8490365
COPIES:	

We reviewed the Technical Memorandum – Draft, provided to the City of Monmouth by Kennedy/Jenks Consultants (K/J), regarding a potential Joint Use Effluent Reuse Facility. Their assistance in helping to frame this option is appreciated and offers additional insight into technical and financial factors that are associated with the option of developing a Joint Effluent Reuse Facility. In the following paragraphs we offer additional comments and in some instances recommend modifications or refinement to the cost apportionment and "buy-in" methodology. The purpose of this review and our comments is to offer both City's and their respective consultant teams the benefit of our history with the Independence system, and address as many uncertainties as possible if this option is to advance to a final intergovernmental agreement (IGA).

COST PROPORTIONING

The proposed methodology for sharing the cost of disposing treated effluent from Independence appears to rely on each City paying a portion of the costs for initial capital, and on-going operation and maintenance (O&M). The amount of cost apportioned to each city is prorated according to the total volume of effluent that is land applied. A proration based on total effluent applied appears reasonable and fair. The areas that we recommend clarification on include:

- The share of the initial capital cost paid by Independence should take into account the actual cost and residual value of capital investment developing the system as of the date of an agreement. A final agreement should include a detailed itemization of costs to date, including a breakdown of equipment, land, professional services, and if applicable income. Mechanical equipment that remains in use has some residual "useful life" but is not new. Land in some instances retains its original value.

- Buy-In costs should reflect the remaining useful life of the systems mechanical, electrical and structural components discounted for the years of operation and associated wear and tear that has occurred since the Reuse Facilities were constructed.

- Using an "estimated average rate per million gallons (Mgal)" as a buy in cost is valuable for preliminary estimates to buy into and share the on-going O&M. However, the method described in the February 8, 2013 K/J memorandum appears to rely on the final proportional share (38.4%) and does not take into account the initial proportion attributable to Independence (18%). We recommend a buy-in rate that applies an average of the initial and estimated future share of use.

- Rather than locking in at an estimated average rate over a 20-year period, on-going O&M costs should be paid using the actual proportionate usage each year or month that expenses are incurred. Since these are expenses that are reported monthly it should be easy to track and pay the total amount based on the recorded effluent that is land applied.

RECOMMENDED APPROACH

Modify the Buy-In Amount to Reflect Residual Service Life

Modify the estimated "buy-in" amount that is intended to compensate for the existing facilities to reflect the fact that these will have been in use for approximately 12 years (2002 to 2014) out of a 30 year service life. Independence did not benefit from these facilities for the initial 12 years but should pay for prorated share of remaining service life. Independence and Monmouth will share the cost for future facilities and upgrades. The total capital cost for the original facility of $3,416,000, as reported in the K/J memorandum, has been assumed to reduce in useful life on a straight-line basis with a total 30-year service life. Applying this approach indicates a 60% residual service life (18-years/30-years) which produces a total residual value in the year 2014 of $2,049,600.

Modify the Method of Proportioning Capital Cost to Reflect Average Usage

The proportion of total effluent sent to the reuse facilities by Independence will change over time, and is projected to increase from 18% in 2014 to 38.4% in 2035. If initial "buy-in" of capital improvements is negotiated based on projected percentage of total effluent discharged, it is seems reasonable to use an average of an agreed upon range. The average of Kennedy/Jenks estimates is 28.2%. The actual initial and estimated usage should be updated to reflect the flows identified in the Final Facilities Plan adopted by each city.

In the following table we have adjusted the estimated prorated cost share using the capital costs reported in the J/K memorandum. The adjusted estimates apply the Residual Service Life and Average Effluent noted in the previous paragraphs.

Table 1 – Estimated Capital Cost Share for Joint Reuse Facility

	Estimated Cost	Estimated Independence Share [2]	Rounded Estimated Independence Share
Estimated Total Cost of Existing Facilities [1]	$ 2,049,600	$ 577,987	$ 578,000
Present Worth of Effluent Reuse Pump Station	$ 164,062	$ 46,265	$ 46,270
Present Worth of Effluent Reuse Plan Update	$ 132,812	$ 37,452	$ 37,460
Present Worth of Future of Effluent Facilities	$ 1,033,854	$ 291,546	$ 291,600
Total Estimated Cost	**$ 3,380,328**	**$ 953,250**	**$ 953,330**

(1) Assumes original construction completed and operation initiated in 2002: 12 years operational, 30 years total service life.
(2) Estimated Independence Share assumes 28.2% of Estimated Capital Cost.

Acknowledge Regional Value of Pressure Line Connecting Both Lagoon Systems

There is value to both communities to constructing a forcemain connecting to two lagoon systems. The current approach is to pump and dispose of all treated and disinfected effluent to properties south of Monmouth. Although this arrangement has been well planned and functional, due to the required capital investment and fixed pipe alignments, it does limit future options and negotiations with land owners. Constructing a pressure line connecting the two lagoon systems would allow the flexibility to pump treated effluent either direction. In the future it may be desirable for both communities to have optional or expansion land application sites in the vicinity of the Independence Urban Growth Boundary (UGB). This allows flexibility to adapt to scenarios including but not limited to:

- Dramatic increases in energy and associated pumping costs.
- Changes in sentiment of land owners regarding continued usage as a land application site. In the event the current agreements for land application with private landowners are not renewed or canceled, conveyance to new spray irrigation sites within or immediately adjacent to the Independence UGB is a reasonable alternative for both cities.
- Redundancy in the event of mechanical or pipeline failure, or natural disaster.

If the pipe diameter is selected to convey only treated effluent from Independence to Monmouth it will likely be 8-inches in diameter. To use the pipe under a scenario that conveys future flows from Monmouth to Independence would require a pipe diameter of 12 to 16-inches diameter. The additional cost for the larger pipe diameter is a relatively small portion of the construction cost if included as part of a single project.

We recommend that a buy-in agreement include a shared cost for materials and construction of the pipeline connecting the two lagoon systems. Similar to other capital costs the cost sharing would be proportional to initial and estimated future usage.

Operation and Maintenance Costs

As noted previously we recommend that the O&M costs be based on actual costs and applied on an annual or monthly basis. If an agreement is finalized we recommend that a detailed breakdown be developed which clearly identifies labor, materials and equipment replacement committed to the reuse portion of the treatment process.

APPENDIX G

Itemized Capital Cost Estimates

CONCEPTUAL LEVEL

ALTERNATIVE NO. 1 - LAGOON AERATION RETROFIT

CAPITAL COST

Item	Description	Est. Quant.	Unit	Unit Price	Total Price
1	**HEADWORKS**				
	a. Influent Screen	1	LS	LUMP SUM	$242,250
	b. Grit Removal	1	LS	LUMP SUM	$78,500
2	**LAGOONS**				
	a. Sludge Removal and Disposal Cell Nos. 1 and 2	1	LS	LUMP SUM	$2,450,000
	b. Lagoon No. 1 Transfer Pump Station Replacement	1	LS	LUMP SUM	$108,500
	c. Aerators	80	EA.	$18,300	$1,464,000
	d. Aerator Anchors	80	EA.	$3,700	$296,000
	e. Riprap Dikes	1000	C.Y.	$60.00	$60,000
	f. Perimeter Landscape Improvements	1	LS	LUMP SUM	$72,350
	g. Electrical Power and Controls	1	LS	LUMP SUM	$904,400
	h. Plumbing and Yard Piping	1	LS	LUMP SUM	$24,250
	i. Outlet Modifications	4	EA.	$8,900.00	$35,600
	j. Structural Modifications, Metal, Railings, Concrete Pads	1	LS	LUMP SUM	$30,250
3	**OPERATION AND CONTROL**				
	a. Solids Handeling Structure	1	LS	LUMP SUM	$200,000
	b. Telemetry	1	LS	LUMP SUM	$42,250
	c. Control System	1	LS	LUMP SUM	$42,250
	d. Control Building	1	LS	LUMP SUM	$180,900
4	**EFFLUENT MANAGEMENT**				
	a. Chlorine Contact Chamber Expansion	1	LS	LUMP SUM	$190,000
	b. Flow Paced Chlorine Addition	1	LS	LUMP SUM	$12,250
	c. Chlorine System Replacement	1	LS	LUMP SUM	$72,400
	d. Algae Control System	1	LS	LUMP SUM	$90,450
	e. Dechlorination System	1	LS	LUMP SUM	$114,600
5	**PIPING RECONFIGURATION**				
	a. Outfall Connection	1	LS	LUMP SUM	$48,250

Total Construction Cost	**6,759,450**
Project Soft Costs and Contingency @ 54%	**$3,650,103**
TOTAL COST	**$10,409,553**

ALTERNATIVE NO. 1 - LAGOON AERATION RETROFIT
ESTIMATED ANNUAL OPERATION, MAINTENANCE AND REPLACEMENT COSTS

(2015 Dollars Annual Costs)

Item	Description	Existing 2015
1	**OPERATION AND MAINTENANCE**	
	a. Personnel (i.e. Salary, Benefits, Payroll Tax, Insurance, Training)	$255,200
	b. Administrative Costs (e.g. office supplies, printing, etc.)	$5,820
	c. Insurance	$23,100
	d. Energy Cost (Fuel and/or Electrical)	$120,600
	e. Process Chemicals (Chlorine, etc.)	$9,150
	f. Monitoring & Testing	$8,100
	g. Professional Services	$5,400
	h. Miscellaneous	$550
		$427,920
2	**REPLACEMENT**	
	a. Aerators	$32,500
	b. Electrical Equipment	$5,400
	c. Miscelaneous Mechanical and Plumbing Equipment	$5,400
	d. Controls	$5,400
		$48,700

TOTAL ESTIMATED ANNUAL O, M & R COSTS **$476,620**

CONCEPTUAL LEVEL

ALTERNATIVE NO. 2 - ACTIVATED SLUDGE

CAPITAL COST

Item	Description	Est. Quant.	Unit	Unit Price	Total Price
1	**HEADWORKS**				
	a. Influent Screen	ALL	LS	LUMP SUM	$242,500
	b. Grit Removal	ALL	LS	LUMP SUM	$78,500
	c. Influent Pump Station	ALL	LS	LUMP SUM	$180,900
2	**PROCESS**				
	a. Tanks and Clarifiers	ALL	LS	LUMP SUM	$1,808,900
	b. Process Equipment	ALL	LS	LUMP SUM	$2,960,000
	c. Piping and Mechanical Equipmen	ALL	LS	LUMP SUM	$664,000
	d. Support Structures	ALL	LS	LUMP SUM	$242,000
	e. Electrical Power Improvements	ALL	LS	LUMP SUM	$78,500
	f. Instrumentation and Controls	ALL	LS	LUMP SUM	$114,600
	g. Lagoon No. 1 Transfer Pump Station Replacement	ALL	LS	LUMP SUM	$108,500
3	**OPERATION AND CONTROL**				
	a. Solids Handeling Structure	ALL	LS	LUMP SUM	$150,700
	b. Telemetry	ALL	LS	LUMP SUM	$42,250
	c. Control System	ALL	LS	LUMP SUM	$42,250
	d. Control Building	ALL	LS	LUMP SUM	$180,900
	e. Site Improvements	ALL	LS	LUMP SUM	$150,800
4	**EFFLUENT MANAGEMENT**				
	a. Chlorine Contact Chamber Expansior	1	LS	LUMP SUM	$190,000
	b. Flow Paced Chlorine Additior	1	LS	LUMP SUM	$12,250
	c. Chlorine System Replacement	1	LS	LUMP SUM	$72,400
	d. Algae Control System	1	LS	LUMP SUM	$90,450
	e. Dechlorination System	1	LS	LUMP SUM	$114,600
5	**PIPING RECONFIGURATION**				
	a. Outfall Connection	1	LS	LUMP SUM	$48,250
6	**SOLIDS MANAGEMENT**				
	a. Storage and Settling Tanks	20000	C.Y.	$14.00	$280,000
	b. Liner	20000	S.Y.	$6.50	$130,000
	c. Sludge Processing Equipment	1	LS	LUMP SUM	$1,022,861

Total Construction Cost	**9006111**
Project Soft Costs and Contingency @ 54%	**$4,863,300**
TOTAL COST	**$13,869,411**

ALTERNATIVE NO. 2 - Activated Sludge
ESTIMATED ANNUAL OPERATION, MAINTENANCE AND REPLACEMENT COSTS

(2015 Dollars Annual Costs)

Item	Description	Existing 2015
1	**OPERATION AND MAINTENANCE**	
	a. Personnel (i.e. Salary, Benefits, Payroll Tax, Insurance, Training)	$255,175
	b. Administrative Costs (e.g. office supplies, printing, etc.)	$5,810
	c. Insurance	$23,100
	d. Energy Cost (Fuel and/or Electrical)	$108,000
	e. Process Chemicals (Chlorine, etc.)	$9,150
	f. Monitoring & Testing	$8,075
	g. Professional Services	$5,400
	h. Miscellaneous	$550
		$415,260
2	**REPLACEMENT**	
	a. Blowers	$10,750
	b. Pumps	$16,150
	c. Miscelaneous Mechanical and Plumbing Equipmen	$16,150
	d. Electrical Equipment	$8,075
	e. Controls	$5,390
		$56,515

TOTAL ESTIMATED ANNUAL O, M & R COSTS $471,775

CONCEPTUAL LEVEL

ALTERNATIVE NO. 3 - OXIDATION DITCH

CAPITAL COST

Item	Description	Est. Quant.	Unit	Unit Price	Total Price
1	**HEADWORKS**				
	a. Influent Screen	ALL	LS	LUMP SUM	$242,500
	b. Grit Removal	ALL	LS	LUMP SUM	$78,500
	c. Influent Pump Statior	ALL	LS	LUMP SUM	$180,900
2	**PROCESS**				
	a. Tanks and Clarifiers	ALL	LS	LUMP SUM	$1,809,000
	b. Process Equipment	ALL	LS	LUMP SUM	$2,412,000
	c. Piping and Mechanical Equipmen	ALL	LS	LUMP SUM	$663,250
	d. Support Structures	ALL	LS	LUMP SUM	$241,200
	e. Electrical Power Improvements	ALL	LS	LUMP SUM	$78,400
	f. Instrumentation and Controls	ALL	LS	LUMP SUM	$114,600
	g. Lagoon No. 1 Transfer Pump Station Replacemen	ALL	LS	LUMP SUM	$108,500
3	**OPERATION AND CONTROL**				
	a. Solids Handeling Structure	ALL	LS	LUMP SUM	$150,750
	b. Telemetry	ALL	LS	LUMP SUM	$42,250
	c. Control System	ALL	LS	LUMP SUM	$42,250
	d. Control Building	ALL	LS	LUMP SUM	$180,900
	e. Site Improvements	ALL	LS	LUMP SUM	$150,750
4	**EFFLUENT MANAGEMENT**				
	a. Chlorine Contact Chamber Expansior	1	LS	LUMP SUM	$190,000
	b. Flow Paced Chlorine Addition	1	LS	LUMP SUM	$12,250
	c. Chlorine System Replacement	1	LS	LUMP SUM	$72,400
	f. Algae Control System	1	LS	LUMP SUM	$90,450
	g. Dechlorination System	1	LS	LUMP SUM	$114,600
5	**PIPING RECONFIGURATION**				
	a. Outfall Connection	1	LS	LUMP SUM	$48,250
6	**SOLIDS MANAGEMENT**				
	a. Storage and Settling Tanks	20000	C.Y.	$14.00	$280,000
	b. Liner	20000	S.Y.	$6.50	$130,000
	c. Sludge Processing Equipment	1	LS	LUMP SUM	$1,022,875

Total Construction Cost	**$8,456,575**
Project Soft Costs and Contingency @ 54%	**$4,566,551**
TOTAL COST	**$13,023,126**

ALTERNATIVE NO. 3 - OXIDATION DITCH
ESTIMATED ANNUAL OPERATION, MAINTENANCE AND REPLACEMENT COSTS

(2015 Dollars Annual Costs)

Item	Description	Existing 2015
1	**OPERATION AND MAINTENANCE**	
	a. Personnel (i.e. Salary, Benefits, Payroll Tax, Insurance, Training)	$255,200
	b. Administrative Costs (e.g. office supplies, printing, etc.	$5,810
	c. Insurance	$23,100
	d. Energy Cost (Fuel and/or Electrical)	$102,250
	e. Process Chemicals (Chlorine, etc.)	$9,150
	f. Monitoring & Testing	$8,075
	g. Professional Services	$5,400
	h. Miscellaneous	$550
		$409,535
2	**REPLACEMENT**	
	a. Brushes	$21,500
	b. Pumps	$16,150
	c. Miscelaneous Mechanical and Plumbing Equipmen	$16,150
	d. Electrical Equipment	$10,800
	e. Controls	$5,500
		$70,100

TOTAL ESTIMATED ANNUAL O, M & R COSTS **$479,635**

CONCEPTUAL LEVEL

ALTERNATIVE NO. 4 - SEQUENCING BATCH REACTOR (SBR)

CAPITAL COST

Item	Description	Est. Quant.	Unit	Unit Price	Total Price
1	**HEADWORKS**				
	a. Influent Screen	ALL	LS	LUMP SUM	$242,250
	b. Grit Removal	ALL	LS	LUMP SUM	$78,500
	c. Influent Pump Station	ALL	LS	LUMP SUM	$180,900
2	**PROCESS**				
	a. Tanks	ALL	LS	LUMP SUM	$1,125,000
	b. Process Equipment	ALL	LS	LUMP SUM	$2,420,000
	c. Piping and Mechanical Equipmen	ALL	LS	LUMP SUM	$950,000
	d. Support Structures	ALL	LS	LUMP SUM	$241,200
	e. Electrical Power Improvements	ALL	LS	LUMP SUM	$78,400
	f. Instrumentation and Controls	ALL	LS	LUMP SUM	$211,100
	g. Lagoon No. 1 Transfer Pump Station Replacement	ALL	LS	LUMP SUM	$108,500
3	**OPERATION AND CONTROL**				
	a. Solids Handeling Structure	ALL	LS	LUMP SUM	$150,750
	b. Telemetry	ALL	LS	LUMP SUM	$42,250
	c. Control System	ALL	LS	LUMP SUM	$42,250
	d. Control Building	ALL	LS	LUMP SUM	$180,900
	e. Site Improvements	ALL	LS	LUMP SUM	$150,750
4	**EFFLUENT MANAGEMENT**				
	a. Chlorine Contact Chamber Expansior	1	LS	LUMP SUM	$190,000
	b. Flow Paced Chlorine Additior	1	LS	LUMP SUM	$12,500
	c. Chlorine System Replacement	1	LS	LUMP SUM	$72,400
	d. Algae Control System	1	LS	LUMP SUM	$90,450
	e. Dechlorination System	1	LS	LUMP SUM	$114,600
5	**PIPING RECONFIGURATION**				
	a. Outfall Connection	1	LS	LUMP SUM	$48,250
6	**SOLIDS MANAGEMENT**				
	a. Storage and Settling Tanks	20000	C.Y.	$14.00	$280,000
	b. Liner	20000	S.Y.	$6.50	$130,000
	c. Sludge Processing Equipment	1	LS	LUMP SUM	$807,500

Total Construction Cost	**$7,948,450**
Project Soft Costs and Contingency @ 54%	**$4,292,163**
TOTAL COST	**$12,240,613**

ALTERNATIVE NO. 4 - SEQUENCING BATCH REACTOR
ESTIMATED ANNUAL OPERATION, MAINTENANCE AND REPLACEMENT COSTS

(2015 Dollars Annual Costs)

Item	Description	Existing 2015
1	**OPERATION AND MAINTENANCE**	
	a. Personnel (i.e. Salary, Benefits, Payroll Tax, Insurance, Training)	$255,200
	b. Administrative Costs (e.g. office supplies, printing, etc.)	$5,810
	c. Insurance	$23,100
	d. Energy Cost (Fuel and/or Electrical)	$88,300
	e. Process Chemicals (Chlorine, etc.)	$9,150
	f. Monitoring & Testing	$8,010
	g. Professional Services	$5,400
	h. Miscellaneous	$550
		$395,520
2	**REPLACEMENT**	
	a. Blowers	$8,100
	b. Pumps	$16,150
	c. Miscelaneous Mechanical and Plumbing Equipmen	$10,800
	d. Electrical Equipment	$10,800
	e. Controls	$5,400
		$51,250

TOTAL ESTIMATED ANNUAL O, M & R COSTS **$446,770**

CONCEPTUAL LEVEL

ALTERNATIVE NO. 5 - MEMBRANE BIOREACTOR (MBR)

CAPITAL COST

Item	Description	Est. Quant.	Unit	Unit Price	Total Price
1	**HEADWORKS**				
	a. Influent Screen	ALL	LS	LUMP SUM	$242,500
	b. Grit Removal	ALL	LS	LUMP SUM	$102,500
	c. Influent Pump Statior	ALL	LS	LUMP SUM	$180,900
2	**PROCESS**				
	a. Tanks	ALL	LS	LUMP SUM	$1,125,000
	b. Process Equipment	ALL	LS	LUMP SUM	$1,650,000
	c. Piping and Mechanical Equipmen	ALL	LS	LUMP SUM	$950,000
	d. Support Structures	ALL	LS	LUMP SUM	$241,200
	e. Electrical Power Improvements	ALL	LS	LUMP SUM	$78,400
	f. Instrumentation and Controls	ALL	LS	LUMP SUM	$211,100
	g. Lagoon No. 1 Transfer Pump Station Replacemen	ALL	LS	LUMP SUM	$108,600
3	**OPERATION AND CONTROL**				
	a. Solids Handeling Structure	ALL	LS	LUMP SUM	$150,750
	b. Telemetry	ALL	LS	LUMP SUM	$42,250
	c. Control System	ALL	LS	LUMP SUM	$42,250
	d. Control Building	ALL	LS	LUMP SUM	$180,900
	e. Site Improvements	ALL	LS	LUMP SUM	$150,750
4	**EFFLUENT MANAGEMENT**				
	a. Chlorine Contact Chamber Expansior	ALL	LS	LUMP SUM	$190,000
	b. Flow Paced Chlorine Addition	ALL	LS	LUMP SUM	$12,250
	c. Chlorine System Replacement	ALL	LS	LUMP SUM	$72,400
	d. Algae Control System	ALL	LS	LUMP SUM	$90,450
	e. Dechlorination System	ALL	LS	LUMP SUM	$114,600
5	**PIPING RECONFIGURATION**				
	a. Outfall Connection	ALL	LS	LUMP SUM	$48,250
6	**SOLIDS MANAGEMENT**				
	a. Storage and Settling Tanks	20000	C.Y.	$14.00	$280,000
	b. Liner	20000	S.Y.	$6.50	$130,000
	c. Sludge Processing Equipment	ALL	LS	LUMP SUM	$807,500

Total Construction Cost	**$7,202,550**
Project Soft Costs and Contingency @ 54%	**$3,889,377**
TOTAL COST	**$11,091,927**

ALTERNATIVE NO. 5 - MEMBRANE BIOREACTOR
ESTIMATED ANNUAL OPERATION, MAINTENANCE AND REPLACEMENT COSTS

(2015 Dollars Annual Costs)

Item	Description	Existing 2015
1	**OPERATION AND MAINTENANCE**	
	a. Personnel (i.e. Salary, Benefits, Payroll Tax, Insurance, Training)	$255,200
	b. Administrative Costs (e.g. office supplies, printing, etc.	$5,810
	c. Insurance	$23,100
	d. Energy Cost (Fuel and/or Electrical)	$92,350
	e. Process Chemicals (Chlorine, etc.)	$9,250
	f. Monitoring & Testing	$8,100
	g. Professional Services	$5,400
	h. Miscellaneous	$550
		$399,760
2	**REPLACEMENT**	
	a. Blowers	$8,100
	b. Pumps	$16,150
	c. Membrane Filters	$10,750
	d. Miscelaneous Mechanical and Plumbing Equipmen	$10,750
	e. Electrical Equipment	$10,750
	f. Controls	$8,100
		$64,600

TOTAL ESTIMATED ANNUAL O, M & R COSTS **$464,360**

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