City of Veneta Wastewater System Master Plan & Capital Improvement Plan

April 2009

Weber Elliott Engineers, P.C.

Partners in Quality and Commitment.

P. O. Box 10145 ▾ Eugene, OR 97440 ▾ (541) 461-9779 ▾ fax (541) 461-9780 ▾ www.weberelliott.com

WASTEWATER SYSTEM MASTER PLAN and CAPITAL IMPROVEMENT PLAN

Prepared for:
City of Veneta
P.O. Box 458
Veneta, OR 97487

Prepared By:

Weber Elliott Engineers, P.C.
P.O. Box 10145
Eugene, OR 97440

April 2009

PROJECT TEAM:

City Administrator	Ric Ingham
Public Works Director	Kyle Schauer
Community Services Director	Brian Issa
Council (upon work authorization)	Tim Brooker – Mayor Marion Esty Thomas Cotter Darrell Carman Sharon Hobart-Hardin
Project Manager	Jerry T. Elliott, P.E.
Project Engineer	Paul T. Hadnutt, P.E.
Modeling Technician	Alex Lowe

(This page intentionally blank)

TABLE OF CONTENTS

ELEMENT ***PAGE***

TABLE OF CONTENTS iii

I. EXECUTIVE SUMMARY ..... a

1. INTRODUCTION 1-1
- **1.1** Background 1-1
- **1.2** Scope of Work 1-1
  - **1.2.1** Task 1 – System Information Review 1-2
  - **1.2.2** Task 2 – Hydraulic Model Development 1-2
    - Hydraulic Modeling 1-2
    - Model Calibration 1-3
    - Infiltration/Inflow 1-3
    - System Constraints & Deficiencies Identification 1-3
  - **1.2.3** Task 3 – Systems Alternative Analysis 1-4
    - Collection System Alternatives 1-4
    - Pump Stations 1-4
    - Future Collection System Analysis 1-4
    - Effluent Management Alternatives 1-5
    - Level IV Treatment 1-5
  - **1.2.4** Task 4 – Final Report 1-6
    - Capital Improvement Plan 1-6
    - Draft Report Public Review & Council Presentation 1-6
- **1.3** Final Report 1-7

2. STUDY AREA CHARACTERISTICS 2-1
- Chapter Summary 2-1
- **2.1** Study Area 2-1
- **2.2** Physical Environment 2-1
- **2.3** Socio-Economic Environment 2-2
  - **2.3.1** Economic Conditions and Trends 2-2
  - **2.3.2** Population 2-2

3. COLLECTION SYSTEM 3-1
- Chapter Summary 3-1
- **3.1** Existing System Description 3-3
  - **3.1.1** Gravity System 3-3
  - **3.1.2** Pressure System 3-3

Lift Stations 3-3
Pressure Pipes 3-5
3.1.3 System Conditions 3-5
3.2 Sewer System Analysis 3-6
3.2.1 Hydraulic Model 3-6
3.2.2 Phase 1: Model Dimensions 3-7
3.2.3 Phase 2: Collection System Layout 3-7
3.2.4 Phase 3: Assembling the Model 3-7
3.2.5 Phase 4: Establishing Loading Data 3-7
Population 3-8
Sewer Sheds 3-8
Flow Monitoring 3-8
Waste Water Treatment Facility Records 3-8
Selected Storm Loading 3-9
3.2.6 Phase 5: Model Calibration 3-10
3.2.7 Phase 6: Model Run and Presentation 3-10
3.3 Infiltration and Inflow (I/I) 3-11
3.4 Current Deficiencies 3-12
3.5 Projected Deficiencies 3-14
3.6 Alternative Solutions 3-14
Alternative Design Overview & Definitions 3-14
3.6.1 Common Projects 3-15
3.6.2 Alternative 1 3-15
Concept Description 3-15
Construction Elements 3-15
3.6.3 Alternative 2 3-16
Concept Description 3-16
Construction Elements 3-16
3.6.4 Alternative 3 3-17
Concept Description 3-17
Construction Elements 3-17
3.6.5 Alternative 4 3-18
Concept Description 3-18
Construction Elements 3-19
3.6.6 Alternative 5 3-19
Concept Description 3-19
Construction Elements 3-19
3.7 Alternatives Analysis 3-20
Environmental Concerns 3-20
Environmental Risks 3-20
Energy Usage 3-20
Service Requirements and Reliability 3-20
Social Concerns 3-21
Construction 3-21
Land Acquisition and Easements 3-21

Alternative 1 3-22
Environmental 3-22
Social 3-22
Alternative 2 3-22
Environmental 3-22
Social 3-23
Alternative 3 3-23
Environmental 3-23
Social 3-24
Alternative 4 3-24
Alternative 5 3-24
Environmental 3-24
Social 3-24
3.8 Cost Estimates of Alternatives 3-25
3.8.1 Construction Costs 3-25
3.8.2 Operation & Maintenance Cost 3-26
3.8.3 Cost Comparison 3-26
3.9 Selected Alternative 3-27
3.10 Glossary 3-27
Pipe Bursting 3-27
Nominal Diameters 3-28
Force Mains 3-28
Pipe Abandonment 3-28
Service Lateral 3-28
Manhole 3-28
Water Pump 3-28
Bypass Pump 3-28
Right of Way 3-28
Easement 3-28
Gravity Pipe Upsizing 3-28

4. REGULATORY CRITERIA 4-1
Chapter Summary 4-1
4.1 Regulatory Criteria 4-1
4.1.1 Organic & Sediment Load (BOD & TSS) 4-2
4.1.2 Bacterial Limits 4-3
4.1.3 General Suitability Criteria 4-3
4.1.4 Other Permit Requirements 4-3
4.1.5 Biosolids Management 4-4
4.1.6 Annual Reports 4-4
4.2 Future Regulations 4-4
4.2.1 Discharge Standards 4-5
4.2.2 Plant Performance 4-5
4.2.3 Additional Parameters 4-6
4.3 Water Reuse Regulations 4-7

5. TREATMENT PROCESSES 5-1
Chapter Summary 5-1
5.1 Existing System Basis of Design 5-2
5.1.1 Headworks 5-2
5.1.1.1 Influent Flow Measurement 5-3
5.1.1.2 Influent Pump Station 5-3
5.1.1.3 Surge Basin Current Conditions 5-4
Capacity Analysis 5-4
5.1.1.4 Screening & Grit Removal 5-5
5.1.1.5 Flow Splitting 5-6
5.1.2 Aeration System 5-7
5.1.2.1 Input Loads 5-7
5.1.2.2 Plant Performance 5-8
5.1.3 Clarifiers 5-10
5.1.4 Disinfection 5-10
5.1.5 Effluent Management 5-12
5.1.5.1 Water Recycle System 5-12
5.1.5.2 Effluent Flume 5-12
5.1.5.3 River Discharge 5-12
5.1.5.4 Irrigation Storage 5-12
5.1.5.5 Land Application Reuse Area 5-13
5.1.6 Sludge Management 5-16
5.1.6.1 Facultative Sludge Lagoons 5-16
5.1.6.2 Sludge Dredge 5-16
5.2 Future Plant Concept 5-16
5.2.1 Headworks 5-17
5.2.1.1 Influent Flow Measurement 5-17
5.2.1.2 Influent Screw Pumps 5-17
5.2.1.3 Surge Basin 5-18
5.2.1.4 Screening 5-18
5.2.1.5 Flow Splitting 5-18
Headworks Summary 5-19
5.2.2 Aeration and Clarifiers 5-19
5.2.3 Facultative Sludge Lagoon 5-19
5.2.4 U.V. Disinfection 5-19
5.2.5 Reuse Upgrades 5-20
5.2.6 Irrigation Pump Upgrade 5-20
5.2.7 River Outfall 5-20
5.3 Wastewater Treatment Plant Costs 5-20

6. WATER REUSE 6-1
Chapter Summary 6-1
6.1 Background 6-1
6.2 Reuse Required 6-2

6.3 Reuse Alternatives 6-4
6.3.1 Alternative 1: Secondary Effluent Reuse 6-4
Nitrogen 6-4
Soil Uptake Capacity 6-6
6.3.2 Alternative 2: Class "A" Reuse 6-7
6.4 Technology Requirements for Class "A" Reuse 6-8
6.5 Reuse Facilities Size & Costs 6-9
6.5.1 Processes 6-9
6.5.2 Distribution 6-9
6.5.3 Costs 6-10
6.5.4 Class "A" Technology – Operation and Maintenance 6-11
6.6 Economic Assessment of Reuse 6-12
6.6.1 Water Resource Development Costs 6-12
6.7 Recommendation 6-13

7. CAPITAL IMPROVEMENT PLAN 7-1
Chapter Summary 7-1
7.1 Capital Improvement Plan 7-1
7.1.1 Collection System Elements 7-1
Project C1-2009 7-1
Project C2-2010 7-2
Project C3-2013 7-2
Project C4-2017 7-2
Project C5-2021 7-3
Project C6-2017 7-3
Project C7-2015 7-3
Project C8- 2021 7-3
Project C9- 2021 7-4
Project C10-2013 7-4
Project C11-2013 7-4
Project C12-2012 7-4
Project C13-2029 7-4
7.1.2 Treatment Process Elements 7-5
Project T1-2009 7-5
Project T2-2010 7-5
Project T3-2011 7-6
Project T4-2011 7-6
Project T5-2012 7-6
Project T6-2010 7-6
Project T7-2018 7-6
7.1.3 Water Reuse Project Elements 7-6
Project R1-2010 7-7
Project R2-2011 7-7
Project R3-2015 7-7
Project R4-2020 7-7

Project R5-2017 7-7

APPENDICES

A. Study Area Characteristics – From 1997 Facility Plan
B. For Use with Chapter 3
C. Treatment Process Improvements Cost Estimate

LIST OF TABLES

EX1 Capital Improvement Plan

3.1a Pipe Material & Size Comparison Data
3.1b WestLane Center Lift Pump Station Data
3.1c Pine Street Lift Station Data
3.3c October 3, 2008 – Storm Flow Measure
3.4a Current Model Pipe Deficiencies
3.4b Current Model Manhole Deficiencies
3.7 Alternative Comparison Summary
3.8 Alternative Cost Comparison

4.1a Winter Discharge Permit Limit
4.1b May/October Discharge Permit Limit
4.1c Bacteria Discharge Permit Limit
4.2a Comparison of Total Suspended Solids Treatment Efficiency Requirements
4.2b Effluent TSS Required (mg/L) to Meet Discharge Limit

5.1a Existing Plant Process Design Parameters
5.1b Surge Basin Capacity Requirements 2007
5.1c Influent Load
5.1d Effluent Discharge Winter Month Average
5.1e Effluent Transition Month Average
5.1f UV Design Parameters
5.1g Assessment of Current Land Requirements
5.1h Land Requirements If All Poplars
5.2a 2030 Pump Capacity
5.2b Summary of Headworks Costs
5.3a Summary of Treatment Process Costs

6.1a Acceptable Uses for Various Wastewater Treatment Classes
6.3a Existing Land Application Area
6.3b Nitrogen Sampling Results
6.5a Class “A” Estimate of Cost
6.5b Class “A” Estimate of Added Annual Operation Cost

7.1 Capital Improvement Plan

LIST OF FIGURES

3.1 Veneta Sewer Shed Map
3.2a Flow Comparison
3.2b Model Outfall vs. Peak Hour
3.3a Peak Day Infiltration and Inflow
3.5 2030 Sewer Concept: Alternative 1
3.6 2030 Sewer Concept Alternative 2
3.7 2030 Sewer Concept Alternative 3
3.8 2030 Sewer Concept Alterative 5

5.1 Existing Treatment Plant Schematic
5.2 Future Plant Layout

6.1 Water Reuse Distribution

EXECUTIVE SUMMARY

Study Overview and Objectives:

The City of Veneta authorized this engineering study to provide planning guidance for City operated wastewater collection and treatment facilities. The study emphasizes collection system analysis but also examines the expected service life and performance of the treatment processes.

The study objectives are:

- Evaluate existing and future service loads to the wastewater collection system
- Recommend a wastewater collection system alternative that provides for management of future service growth
- Define capital improvements required to provide for the City's wastewater collection and treatment system through to year 2030
- Provide budget costs estimates for the identified improvements
- Provide guidance and recommendations relative to continued management and development of wastewater infrastructure

Wastewater Collection System:

A hydraulic model was developed to predict the collection system performance under differing service conditions. Five collection system alternatives were evaluated using the model and a recommended alternative was selected that provides that most of the future growth within the city is accommodated by a new collection system pump network that routes increased sewer flow around the City. Full implementation of the selected collection system alternative will require seven years and costs an estimated $4.1 million.

Treatment System:

Analysis of the existing treatment facilities indicates that they will begin to experience hydraulic and process overloads as early as 2014. The recommended plan calls for changes to the influent pumping and screening systems to accommodate the changes recommended in the collection system plan. The treatment plant processes will need to be increased by a factor of about two to accommodate anticipated growth through the service year of 2030. Plant upgrades include processes and disinfection. After the plant upgrade is complete and scheduled in year 2017, the treatment technology will need to be changed to include a tertiary treatment process. The tertiary processes will ensure that the City does not exceed its mandated wastewater load to the Long Tom River.

Water Reuse:

Changes in the Oregon regulations applicable to the land application system alter the design premise for the land application system and mean that no immediate changes in the land application system are required to accommodate service needs

through 2030. The study included an analysis of additional requirements to provide for direct reuse within the City. A basic distribution system was reviewed and the costs of system development were compared against the cost of development of a potable water source. The conclusion is that, at this time, development of a full water reuse system is not cost effective. However, future economic changes and dynamics relative to water reuse may alter this evaluation.

Capital Improvement Plan:

The Capital Improvement Plan cost and schedule is provided in Table Ex1. More comprehensive descriptions of the project elements can be found in the report text and Chapter 7 provides a full summary.

	Collection System																						
C1	Upsize to new 21" modified gravity pipe under Hunter/Territorial intersection	**$282,800**																					
C2	Upsize to new 10", 12", 15" gravity pipe under Territorial south of Hunter from MH I-5 to MH Ba-6	**$542,300**																					
C3	Install new 12" pressure line from Waste Water Treatment Facility to Westlane forcemain				**$510,500**																		
C4	Install new 12" pressure line from Westlane forcemain to Hwy 126 near railroad									**$154,300**													
C5	Install new 12" pressure line from Hwy 126 near railroad to Hunter													**$148,900**									
C6	New North Pump Station north of Todd Way near shore of Fern Ridge Resevoir, includes 4" forcemain									**$559,800**													
C7	New East Pump Station 800 feet south of Hunter off Huston, includes new 8" forcemain and 8" or 12" gravity							**$1,356,800**															
C8	Install new 8" forcemain extension for New East Pump Station													**$39,400**									
C9	Install new 12" forcemain extension for Pine Street Pump Station													**$34,700**									
C10	Replace pumps at Westlane station					**$32,600**																	
C11	Replace pumps at Pine Street station								**$59,700**														
C12	Main Pump Control Center (SCADA Brain)				**$46,400**																		
C13	Common Projects: upsize gravity pipe under Parkside, 8th, Cheney and Oak Island from 8" to 10" or 12"																			**$352,900**			
	Wastewater Treatment																						
T1	Aeration Pipe and W. Hunter flowmeter	**$21,300**																					
T2	Headworks Preliminary Engineering		**$58,000**																				
T3	Headworks & Screening			**$617,000**																			
T4	Modify FSL				**$736,000**																		
T5	Biolac Expansion					**$2,083,000**																	
T6	UV System Changes	**$391,100**																					
T7	Process Design										**$697,200**												
	Water Re-use																						
R1	Poplar Tract Assessing		**$7,000**																				
R2	Poplar Harvest and Replant				**$36,000**		**$36,000**																
R3	Irrigation Pump Upgrade							**$87,700**															
R4	U.V. System Changes - Class A											**$270,000**											
R5	Reuse Distributrion							**$50,000**					**$460,000**										
		$1,302,500		**$1,945,900**		**$2,151,600**		**$1,554,200**		**$1,411,300**		**$730,000**		**$223,000**		**$0**		**$0**		**$352,900**		**$0**	

LEGEND:

2008 Value

* All figures include design, construction and contingency costs

City of Veneta

Wastewater Master Plan

Table 7.1

WEBER ELLIOTT ENGINEERS, P.C.

1. INTRODUCTION

1.1 - Background

The City of Veneta has experienced unprecedented growth since lifting the sewer moratorium and construction of a new wastewater treatment plant in 2001. The City's 1997 Wastewater Facilities plan projected a year 2007 service population of 4071. The current service population is in excess of 4200 and continues to grow. The high wastewater flow rates caused by the City's growth and groundwater infiltration fueled the City's leadership to request the development of a revised Wastewater System Master Plan (Master Plan).

The City of Veneta's last Wastewater Master Plan was prepared in November 1997. It was adopted by Council and approved by the Oregon Department of Environmental Quality.

The City's RFP states that the Master Plan shall meet Oregon Administrative Rule (OAR) 340. This OAR applies to *Facility Plans,* generated in response to compliance orders or permit violations and subject to Oregon Department of Environmental Quality (DEQ) approval. The City is not under any compliance orders from the Department and therefore the DEQ may not review or approve this planning document. The City's existing Facility Plan will remain valid through the planning period (2021) or until permit conditions dictate a facility change.

This Master Plan will serve as the City's planning and budgeting guidance document for the interim. As projects from this document are scheduled, DEQ will need to be provided with appropriately scoped Preliminary Engineering Reports based upon the criteria established in this plan and the interim project definitions. DEQ approval of specific projects will be based upon review of the Preliminary Engineering Report.

1.2 - Scope of Work

The Scope of Work as submitted by Weber Elliott Engineers, P.C. and agreed to by the City in a contract award letter dated November 27, 2007 is provided in Sections 1.2.1 through 1.2.4. Modification to the Scope of Work or document development is highlighted, and italicized within this document. The work consisted of four tasks:

- System Information Review
- Hydraulic Model Development
- Systems Alternative Analysis
- Final Report

In addition to the development and analysis involved in these work tasks, the recommended alternatives are to be compiled and presented in a Capital Improvement Program.

1.2.1 - Task 1 - System Information Review

Using meetings between City staff and the consultant team, communication chains and protocols were established. Key members of the analysis team attended a kick-off meeting and reviewed project guidelines.

The Urban Growth Boundary (UGB) was used to serve as the Study Area Boundary. We established physical characteristics and land use and growth patterns for the wastewater system. We relied on data from the 1997 mapping and 1997 facility plan as appropriate. We also coordinated land use patterns and densities with the City planners and available studies by others.

1.2.2 - Task 2 - Hydraulic Model Development

Hydraulic Modeling

A hydraulic model was developed of the collection system from a layout of the existing network of pump stations, pipes and manholes. The model was built from the existing sewer layout prepared for Veneta by the Lane Council of Governments. Data was field verified and checked against city held record drawings. Inconsistencies were modified as required.

A commercially available computer based modeling program for hydraulic modeling was used to assess collection system hydraulic performance. The program was capable of assimilating new data and updating the model as the system grows. Section 3 of this report describes the specific inputs and outcome of the modeling tasks.

The model was based on “sewersheds”, a network of pipes and pumps that form a distinct analysis zone. Sewersheds are illustrated in Figure 3.1 of Chapter 3. The following sewersheds were identified in the scope of work:

- Main System (Treatment Plant)
- Pine Street Pump Station System
- Jeans Road System (WestLane Pump Station System)
- Huston System (area east of the Pine Street Pump Station network)
- Northeast system (area east of Jeans Road System)

The system analysis included additional sewers scheduled for installation as part of the Bolton Hill project of summer, 2008.

Data was inserted into a database management program (Microsoft Excel) to allow for data sharing in a variety of computer based formats. Pipeline segments and manholes were given identification codes. The model focused on major trunk lines within the system with emphasis on the following areas:

- Sertic Road
- 8th Street
- West Hunter - through Shady Hollow

- 6th Street
- 7th Street and Hunter Collector
- Territorial Highway
- Downtown core area
- East Hunter System
- Pine Street System (Emphasis on Oak Island)
- North Territorial system including the WestLane Pump station

The hydraulic modeling extended to a cut-off point of 80 to 100 homes. Where it was appropriate or needed for resolution, as few as 25 homes became the cut off point. Zoning and land use maps were used to estimate future flow contributions in various sewer basins.

Model Calibration

Three sewersheds were monitored for three weeks to provide model calibration data. Data loggers were installed to log water depth, time of day and velocity. Data was evaluated to estimate rainfall event response and sewer performance. Parts of other sewersheds were evaluated using existing flow monitoring equipment and pump run hours. Summer storm events were not modeled as these events do not present collection system problems for Veneta.

The scope of work relied on the successful operation and data acquisition from the City's existing influent flow monitoring equipment.

Infiltration/Inflow

Based upon prior research a smoke testing program was not included in the scope. Infiltration and inflow were assessed as part of the model calibration in the scope of work.

System Constraints & Deficiencies Identification

We prepared a deficiencies list that summarized the hydraulic, capacity, design and maintenance limitations identified by the analysis. Collection system bottlenecks and/or deficiencies were illustrated on a system map. Accompanying the map was a spreadsheet that summarized the findings and provided descriptions.

The deficiency summary report allowed for staff and engineers to identify problems and delineate alternatives for resolution. These alternatives would then be evaluated for cost effectiveness and integrated into a Capital Improvement Program with established timelines and phasing.

The City, operators, planners and engineers reached consensus on the nature of issues affecting the facilities. A workshop was held to review deficiencies with these members of the City's management team. An outside QA/QC consultant was provided a draft report to obtain an additional professional perspective.

1.2.3 - Task 3 - Systems Alternative Analysis

Alternatives for resolution of deficiencies were summarized from Task 2 and were analyzed. These tasks were developed for the best long term, lowest cost alternative to resolve the system deficiencies. Alternatives were developed for:

- The collection network (including interface with pumping stations)
- Effluent (Summer and Winter) and
- Level IV Treatment

Collection System Alternatives

The collection system analysis draws heavily on the model output and also integrates analysis recently completed by EGR Associates for sewer planning. Optimal pump station locations were identified given capital costs, force main routes and operation and maintenance requirements for multiple stations. Economic and environmental feasibility of alternatives were outlined.

Pump Stations

For the two existing lift stations, West Lane Center and Pine Street, the pipe characteristics and conditions of the piping and pump efficiencies were investigated. From that investigation, the following analyses were made:

- Identified system data gaps and inconsistencies. Analyzed existing pump performance and efficiencies and determined rehabilitation or maintenance needs
- Analyzed requirements for future lift stations and expansion of existing lift stations to serve future growth within the UGB.
- Determined pump station sizing requirements for future service areas; for new and existing pump station upgrades. The area west of Cornerstone and north of Highway 126 and the area west of the current service extension of East Side Pump station (Hunter Road, Tidball and Huston Road service). The alternative of serving both areas with a single lift station (likely located near Hunter and Huston) or by two lift stations was analyzed.
- Developed maps showing proposed lift station locations and general design criteria that match the modeling analysis provided in the Master Plan.
- Developed budget estimates for the new lift stations.

Future Collection System Analysis

- Collection system extensions were recommended to areas not currently served. The transportation System Plan and planning criteria were used as a general guide for layout
- Required collection system capacity expansions were identified and timeline (phasing) for each of these projects was established.

- Budget estimates for construction were prepared in 2008 dollars for each project.

Effluent Management Alternatives

Poplar plantings, the recommended effluent management process from the 1997 plan, have had mixed success. Wetland management constraints have been onerous and the maintenance requirements have exceeded expectations. This Master Plan has identified the land application requirements through the next planning period. Current operations and limitations of the effluent management system were evaluated.

This work does not include further analysis of mixing zones or stream flow for the Long Tom.

Level IV Treatment

Level IV effluent is recognized by the Oregon DEQ as being treated to the highest standard. The effluent would be used for irrigating City parks and fields. The Level IV water would replace potable water produced by City wells. Weber Elliott Engineers' November 2007 proposal stated that a level IV program would offer advantages of:

- Reduced expansion requirements for the poplar effluent management area
- Less demand on City's drinking water system during peak summer demand
- Optimizes use of an available resource in an environmentally beneficial way

A feasibility study of a Level IV program was included in the work. The permit requirements and costs of Level IV treatment system and a distribution system were evaluated. The distribution system includes the infrastructure required to deliver effluent to Territorial Sports Park fields on the south end of Sixth Street. A cost estimate for the distribution system was included.

An evaluation of the existing Biolac system for nitrogen and perhaps phosphorus management was required. Level IV treatment system alternatives included sand filters, disk filters and cloth filters. Potential disinfection changes were evaluated as part of the Level IV assessment.

DEQ has an ongoing committee responsible for updating and revising advanced treatment standards. Potential revisions may positively impact the feasibility. Subsequent to the issuance of this scope of work, DEQ has altered reuse standards to classes A-D rather than Levels I-IV. These reuse standards are reviewed in Chapter 4 and applied to Veneta's situation in Chapter 6.

The following language was added as an addendum to the original scope of work:

> *Evaluate capital improvements required to the surge basin, headworks and treatment train of the existing Biolac treatment process that are required to manage and treat the estimated population and flows determined in Task 1 and modeled in Task 2. Prepare construction costs estimates for these process additions and incorporate these estimates into the Capital Improvement Program.*

1.2.4 - Task 4 - Final Report

This task included the preparation of the Draft Report and Final Report. The Final Report consisted of the Wastewater System Master Plan and Capital Improvement Plan.

Capital Improvement Plan

The Capital Improvement Plan (CIP) summarized and described the selected alternatives for collection, pumping, effluent management and Level IV (if feasible) elements of the Master Plan.

The recommended plan was a phased approach. We anticipated that the initial effort would emphasize correction of collection system deficiencies and the phasing in of pump stations, effluent management processes and Level IV improvements.

> *Actual contingencies developed in this report are 20% construction cost contingency, 20% engineering and a 5% legal and administrative contingency. These contingencies more accurately represented construction and design cost factors in today's construction market place.*

The CIP budget was based on 2008 construction costs with a 30% contingency and an agreed on inflation factor. Each CIP project was identified by a project number that delineates the category and phasing of the project. The collection system CIP projects were summarized in a table according to their flow basins. The CIP documents included:

- A list identified CIP projects for the 20-year planning period
- We identified projects that are required to meet current problems and those required in anticipation of future growth. We allocated future costs to future population to recommend a Collection System Systems Development Charge
- We estimated changes in rate structure as they applied to the CIP program
- We listed criteria for ranking the projects. Such criteria may be surcharge, imminent pipe collapse, model prediction, new regulations, public health risks, environmental risks, etc
- We prepared a preliminary schedule of project construction according to evaluation criteria established by City staff and the engineer

Draft Report Public Review and Council Presentation

We provided the City with a Draft Master Plan for review. The final Master Plan will be suitable for formal presentation to the City Council and public review. The plan will be presented to the Council following draft review by an independent consultant and the City.

1.3 - Final Report

Incorporate and adopt recommended changes in the draft document and prepare a final wastewater Collection and treatment System Master Plan that includes the following elements:

- Prepare the Draft Master Plan Document that incorporates comments and recommendations provided at the presentation provided in the draft
- Provide 6 copies of the document. Consultant shall retain one copy and provide the other five for City use and archives
- Attend a single public hearing/Council meeting for final adoption of the plan. Explain updates and changes from prior presentations
- Provide City all documents in e-format as a pdf document. The sewer model shall be retained at Weber Elliott as discussed with Public Works. E-format shall be provided on compact disk(s)

2. STUDY AREA CHARACTERISTICS

Chapter Summary

This chapter provides the physical and socio-economic setting for Veneta. Very little of the physical environment is reviewed in this document as prior planning documents provide adequate background on physical features. The chapter is organized into the following sections:

2.1 Study Area

2.2 Physical Environment

2.3 Socio-Economic Environment

Supplementary information for this chapter is provided in Appendix A.

Key findings within this chapter are that Veneta lays on top of mostly clay soils, the city has experienced significant growth from 2001 through 2007 and can expect to experience about average (relative to the region) population growth in the immediate future.

2.1 - Study Area

The study area for this project is limited to the Urban Growth Boundary of the City of Veneta. This is the same study area established in the 1997 Wastewater Master Plan (See Appendix A). Therefore this section will not repeat the basic site and community descriptions indicated in the Appendix A.

2.2 - Physical Environment

The 1997 Facilities Plan reviews elements of the physical environment. Those elements are:

- Climate
- Soils
- Geologic Hazard
- Public Health Hazards
- Energy Production and Consumption
- Water Resources
- Flora and Fauna
- Air Quality and Noise
- Environmentally Sensitive Areas

These elements of the physical environment have changed little since the 1997 plan. Elements that have experienced some change are reviewed below.

In addition to the review of Public Health Hazards the City, since 1997, has completed significant wastewater upgrades and installed back-up power supplies resulting in a reduction in the risk to public health. The City has also significantly added to the wastewater collection network which also reduces risk to groundwater supply and public health exposure.

In regard to Water Resources, the City has invested in extensive groundwater supply research programs in year 2000 and subsequently in 2005 through 2007. Those studies have resulted in the addition of two water supply wells into the City's water system. A third well, designated Well 12 is poised to come on line in 2009.

2.3 - Socio-Economic Environment

The Socio-economic environment includes the following elements:

- Economic Conditions and Trends and
- Population

2.3.1 – Economic Conditions and Trends

Many of the economic conditions and trends outlined in the 1997 plan continue to influence the growth and dynamics of Veneta. The City's vision is to provide housing for industrial and commercial developments in neighboring Eugene/Springfield, as well as create jobs close to town.

Veneta remains an attractive location for raising a family and providing a home. As stated in the 1997 plan the City has experienced significant residential growth due to factors such as reasonable commuting distance, sufficient commercial opportunities, competitive housing costs and school system.

It has continued to be a focus of the City's leadership to attract significant local economic development. The City has added the Cornerstone Project, which makes way for industrial and commercial interests, and continues to hold 135 acres zoned for commercial/industrial development. The City has succeeded in attracting some interest including a new "Bi-Mart" store and some small industrial operations. The City really emphasizes its support and desire to attract local employers as part of their economic development agenda.

Housing opportunities have significantly changed in Veneta. The total number of building permits issued has increased each year: 2005, 2006 and 2007 with a slowdown occurring in 2008. The City continues to work hard to provide infrastructure and support for responsible housing development.

2.3.2 – Population

As predicted in the 1997 plan, the lifting of the City's wastewater moratorium and the construction of the wastewater treatment capacity resulted in rapid population growth within the City.

The July 2007 Certified Population provided by Portland State University for Veneta is 4,640. Of this number it is estimated that 122 existing homes are in unsewered portions of the City. Using typical multiplication factors of 2.85 to 2.89 people per home, results in an estimated serviced population of 4230 to 4290.

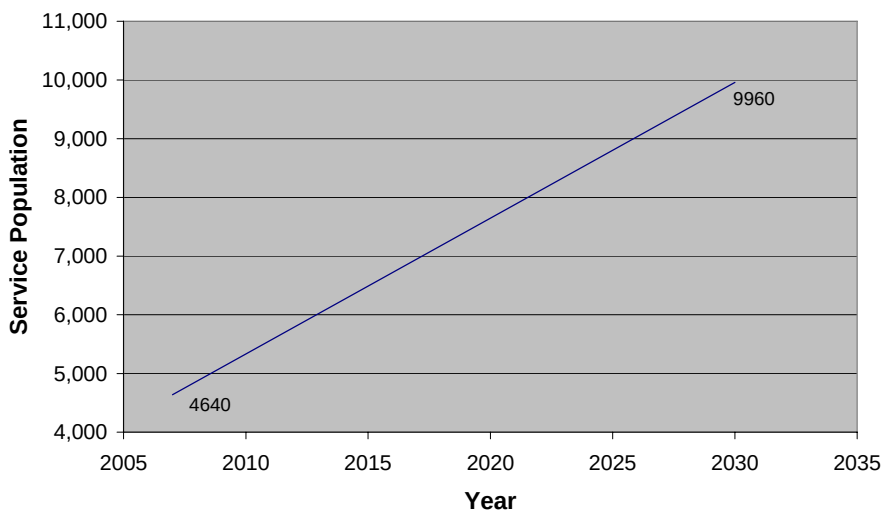
The following figures show the certified population numbers as provided by Portland State.

Year	Population	% Change
2002	3310	N.A.
2003	3480	5.1%
2004	3660	5.1%
2005	3955	8.1%
2006	4240	7.2%
2007	4640	9.4%

Clearly the last three years have resulted in phenomenal growth for the City. The City, as part of its planning efforts has selected to use a population forecast that reflects the rapid growth experienced over the last four years. The growth rate from 2001 has been 3.62 percent and estimated year 2030 population is 12,281 based upon continuation of this rate. (These values based upon LCOG developed "Proposed Coordinated Population Allocations" for 2030 Lane County – Appendix A-2). This growth rate was applied based upon the 2004 adopted 2030 forecast and therefore does not fully reflect the rapid growth experienced in years 2005 through 2007.

Through the development of this report and the associated technical analysis we are assuming that the year 2030 population is within the UGB and that all of the population, at that time, will be connected to the wastewater collection and treatment system. Our analysis and assumptions are based upon a year 2030 service population of 9960 which represents an estimated 91 percent of expected full build-out of the existing UBG. Population and service growth are shown graphically on Figure 2.1a.

Figure 2.1a



As of spring 2008 the City had Planning Commission approval for sufficient additional housing units to accommodate 337 additional residential units. That is, with a current population of 4640 and already approved housing units to provide for over 5500, the City has essentially obligated the treatment plant capacity installed in 2001. Theoretically, at the projected growth rate the original 2021 projected plant capacity will be reached in 2011. This concept of treatment plant capacity is reviewed extensively in Chapter 5.

3. COLLECTION SYSTEM

Chapter Summary

This chapter describes the existing sanitary collection system, identifies existing system weaknesses and deficiencies projected to occur over the 2010-2030 planning period. To identify deficiencies in the collection system, a computer based hydraulic model was developed that showed how the collection system responded to current and future flows. Capital improvement alternatives are developed and analyzed that repair the deficiencies identified. Steps taken to develop the alternatives are explained. The chapter is organized into the following sections:

3.1 Existing System Description

3.2 Sewer System Analysis

3.3 Infiltration and inflow

3.4 Current Deficiencies

3.5 Projected Deficiencies

3.6 Alternative Solutions

3.7 Alternative Analysis

3.8 Cost Estimates of Alternatives

3.9 Selected Alternative

3.10 Glossary

Supplementary information for this chapter is provided in Appendix B.

The collection system analysis begins with a sewered population of 4,300. The base flow during dry weather contributed by each resident in Veneta was calculated to be 70 gallons per capita per day (GPCPD). During the peak hour of the wettest day in winter, the wastewater flow is equivalent to each resident contributing 9.68 times that amount, or 680 GPCPD. Total infiltration and inflow during such day is estimated at 1.8 million gallons. During extreme weather conditions, Public Works staff has observed some manholes overflowing. Most of this extraneous flow is caused by infiltration. These conditions will get worse without correction.

City sewer plans, maps, population figures, treatment plant flow data and independent flow monitors were studied to acquire data for the hydraulic model. The model identified areas of the collection system where pipe capacity was at or beyond its limit. Deficiencies were identified at the intersection of Hunter and Territorial, and points south along Territorial. The model identified surcharged manholes (manholes with flooded floors) on Territorial and Hunter during peak flow hours. The model showed that the four manholes closest to the treatment plant surcharge under normal operating conditions. This is not identified as a hydraulic deficiency. It results from the overflow weir set point that allows overflow to the surge basin while maximizing channel capacity. Overflow has occurred in the

manholes only when the surge basin is so high that the water level interferes with free flow over the weir.

2030 projections were made for population and sewer flow. All growth is expected to be within the existing Urban Growth Boundary (UGB). A population of 9,960 was used to estimate 2030 flow rates. The 2008 per capita flow rates were assumed for the projected 2030 planning period. Our analysis showed that during 2030 loading conditions the collection system experienced locations of sewer overflow.

Initially three sewer design alternatives were developed to correct the deficiencies identified by the model. After review of these alternatives with City staff we were confident that a yet unforeseen solution had eluded the engineers. We then developed two more alternatives to arrive at a solution to providing future service and effectively managing the peak day flows. The alternatives were:

- **Alternative 1** – This is primarily a pressure pipe solution requiring two new pump stations and 16,530 feet of new pressure pipe, where much of the 2030 flow is channeled into a pressure network that bypasses the existing gravity network. It includes the upsizing of 4,950 feet of gravity pipe. The construction costs for Alternative 1 are estimated at $4.20 million. Alternative 1 is illustrated as Figure 3.5.
- **Alternative 2** – This is a solution that focuses on the use of gravity pipe. It calls for one new pump station and 4,920 feet of new pressure pipe. 9,170 feet of existing gravity pipes are upsized (and in some cases bypassed) in key areas to allow for the continued use of the existing gravity system through 2030. Alternative 2 costs an estimated $4.94 million, and is illustrated in Figure 3.6.
- **Alternative 3** – This is also a gravity solution calling for one new pump station and 6,160 feet of new pressure pipe. 6,930 feet of existing gravity pipes are upsized, plus 4,950 feet of new gravity pipe bypasses are constructed. The primary focus of Alternative 3 was to reduce flow to the existing main trunks in Territorial and Hunter so as to reduce the amount of construction within these main and costly corridors. Alternative 3 costs an estimated $5.44 million, and is illustrated in Figure 3.7.
- **Alternative 4** – This is much like Alternative 3 but the focus was to route as much flow north of Hunter by diverting east side lift station flow north to Broadway. After numerous model runs and adjustments it became apparent that because of extensive pipe upgrades in East Broadway this alternative would not be cost effective. An assessment of costs was not made, nor is a graphic representation provided.
- **Alternative 5** – This is a variation of Alternative 4 where all of the future flow from the east is collected at the intersection of Hunter and Territorial and routed north one block and then via a new gravity line west on McCutcheon is sent to the plant. This plan successfully avoids significant construction of the existing main trunk line in Hunter and provides a mostly gravity solution. Alternative 5 is estimated to cost $5.23 million, and is shown in Figure 3.8.

3.1 - Existing System Description

Veneta's collection system is composed of a gravity network and two pump stations. In Table 3.1a, elements of the existing collection system are listed by type. Figure 3.1 shows the locations of Veneta's sewer shed boundaries, existing sewer system features and location of flooded facilities under peak hour flow conditions.

3.1.1 - Gravity System

Most of Veneta's collection system is a gravity system, where sewage is conveyed in sloped pipes. Veneta's collection system contains approximately 60,600 feet of gravity pipe. Table 3.1a shows the number of feet of pipe by material and size.

Table 3.1a

Pipe Material & Size Comparison Data			
Pipe Type	**Pipe size (in inches)**	**Total pipe (in feet)**	**Percent**
Asbestos Cement	8	36,115	59.6
Asbestos Cement	12	857	1.4
Asbestos Cement	15	1,536	2.5
Asbestos Cement	18	1,409	2.3
Asbestos Cement	21	1,560	2.6
PVC	8	12,000	19.8
PVC	10	2,525	4.2
PVC	12	982	1.6
PVC	15	483	0.8
PVC	21	1,733	2.9
PVC	27	1,376	2.3
	Total:	60,576	100

These pipes are three to 44 years old. The oldest pipes are made of asbestos cement (AC or sometimes referred to as Transite). AC pipe is a type of pipe constructed from the 1950's to about 1988. Asbestos fibers were used to provide added strength to the concrete. Although no longer manufactured, buried and functional pipes do not poise an asbestos risk to residents or operators. Some of the AC pipes may be concrete, but the records are not always clear as to type. The rest of the pipes are assumed to be constructed of eight inch polyvinyl chloride (PVC) or AC. Clean-out and laterals were not tabulated and are not a part of this analysis summation.

3.1.2 - Pressure System

Veneta's collection system has two gravity pipe networks, or sewersheds that empty into lift stations. The stations 'lift' sewage from lower elevations up to the main gravity network. Each station consists of a wet well, twin pump, and a length of pipe (or force main) that conveys sewage, under pressure, to the main gravity trunk.

Lift Stations The West Lane lift station serves an area of approximately 120 homes north of Highway 126. It flows into the main trunk at the manhole at Broadway and Territorial Highway. The known operational parameters are summarized in Table 3.1b.

Table 3.1b

WestLane Center Lift Pump Station Data	
Parameter	**Value/Description**
LIFT STATION	
Location	NW corner of Territorial Hwy. & Hwy. 126
Type	Packaged wetwell and duplex drywell
Pump Type	Constant Speed, non-clog
Capacity, each	130 gpm @ 53' TDH
Impeller	8-5/32"
Pump, hp Motors	10 hp, 1750 rpm, 3 phase, 60 Hz, 460V
Level Control Type	Level Transducer with float back-up
Auxiliary Power Type	None
Alarm Type	Audible and visual, on-site – SCADA call out
FORCE MAIN	
Length, Type	1,369' of 6" dia. Transite
Profile	Record drawings not available
Discharge	MH at Broadway & Territorial Hwy.
Storage Capacity	2,013 gallons
Air & Vacuum Release Valves	None

The Pine Street pump station serves approximately 665 homes south of Hunter and east of Territorial Highway. Its flow is discharged into the main trunk at the Hunter Road manhole located at Pine. The operational parameters are summarized in Table 3.1c.

Table 3.1c

Pine Street Lift Station Data	
Parameter	**Value/Description**
LIFT STATION	
Location	SW Corner of Pine Street and Corky Lane
Type	Designed wetwell and duplex drywell
Pump Type	Constant Speed, non-clog
Capacity, each	350 gpm @ 25' TDH
Impeller	9-3/4"
Pump, hp Motors	7.5 hp, 1033 rpm, 3 phase, 60 Hz, 208V
Level Control Type	Level Transducer with float back up
Auxiliary Power Type	Propane engine
Alarm Type	Audible and visual, on-site – SCADA call out
FORCE MAIN	
Length, Type	869' of 10" dia. ductile iron
Profile	Record drawings available
Discharge	MH at Hunter and Pine
Storage Capacity	3,545 gallons
Air & Vacuum Release Valves	None

Pressure Pipes Veneta's collection system has roughly 2,200 feet of pressure pipe (force mains). The location, length, diameter and material of each pressure pipe are listed in Pump Station Tables 3.1b and 3.1c.

3.1.3 - System Conditions

An acceptable sewer system must meet the following three criteria:

1. Adequately protect public health
2. Maintain structural and operational integrity
3. Preserve local environmental resources

Some of Veneta's gravity pipes have been viewed periodically with remotely-operated cameras. Based on viewings in 2005, the condition of the gravity pipe network is considered generally sound, with signs of wear and aging typical of the age of the pipe.

The carrying capacities of some pipes in the gravity network are over their limits, and this has caused two manholes in the network to overflow during peak loads and heavy rain. The manholes are depicted in Figure 3.1. One manhole is near the intersection

of Territorial and Bolton Hill Road, and the second is a couple hundred feet from the plant itself. These overflows violate DEQ regulations. This problem is addressed as part of the sewer system modeling and analysis discussed in Section 3.2. The overflows do present a risk to public health and safety.

3.2 - Sewer System Analysis

This section describes the input parameters, method and results of the collection system analysis. This analysis is the basis of the Sewer System Master Plan and Capital Improvement Plan. The collection system consists of the pipes, manholes, pumps, and wet wells that collect waste from residences, businesses, schools, etc. and convey the sewage to the treatment plant. The treatment system analysis is addressed in Chapter 5.

3.2.1 - Hydraulic Model

The collection system analysis is based on a computer model (hydraulic model) that predicts how the collection system responds to various loading conditions imposed by users and environmental influences (i.e. weather). Flow “patterns” can vary greatly in sewers depending on the season, time of day, and characteristics of influencing storms. Winter months bring rain and high groundwater levels that introduce “peak flows” of five to ten times the amount of water that is experienced in summer months. Residential water use is highest in the morning and evening hours, which contribute to daily flow peaks in the collection system. (Figure 3.2a depicts some actual flow patterns experienced at the treatment plant).

Existing collection system parameters such as pipe size, and slope, etc. are inputs into the model. This model calculates the wastewater depth (level) in the pipes, and in some cases manholes, for various flow patterns. For initial analysis the model input parameters are fixed. That is, the existing pipe layout and structure is set and the model output identifies capacity deficiencies. For future conditions and the examination of alternatives the model elements of slope, length, manhole location and pipe diameter are all manipulated to formulate a workable but smallest size (least cost) alternative.

Various daily flow patterns are input to the model to reflect Veneta’s wastewater flow characteristics. Bentley-Haestad Sewer CAD software was used to simulate the hydraulics of Veneta’s collection system. The Bentley-Haestad software package was selected because of the adaptability of this software program for inclusion from and into other computer programs. Sewer CAD is a stand alone computer modeling package that is user friendly and can be integrated and adapted for inclusion into data base management systems (i.e. Excel) and or geographic information systems (GIS).

The SewerCAD model displays results graphically and in tables. Tables are generated that depict water level in pipes and flow rates. Graphics show pipe profiles, existing ground level, manholes, and wastewater depth.

The development of the model requires a six phase approach:

1. Collecting physical measurements of the system

2. Laying out spatial arrangement of Veneta's collection system in the model
3. Applying the physical measurements to the model layout of the sewer system
4. Calculating loading data to apply to the model
5. Calibrating the model to the field conditions
6. Running the model and collecting the data into maps, graphs, text and tables used to define system deficiencies

3.2.2 - Phase 1: Model Dimensions

Physical dimensions of the collection system, such as pipe diameter, depth, and length, were gathered from the City's record drawings and entered into an Excel spreadsheet. Collected data is tabulated in Appendix B.2.

The pipe lengths used by the model to run its calculations came directly from a scaled .DXF drawing of the sewer system provided by others. For most segments, pipe lengths were also captured from the City's record drawings.

The sewer system plans used to input data into the model were dated from 1976 to 2003 (although the first pipes were actually laid in the 1960's). We noted that any road grade changes made since these plans were drawn may have altered the manhole rim or ground elevations. These changes in rim elevations may not be reflected in the model, because a system survey was outside the scope of this study.

3.2.3 - Phase 2: Collection System Layout

Veneta's collection system layout was prepared by LCOG as a scale .DXF drawing. This layout was cross-referenced and verified between city plans and limited field observations. In many cases, adjustments were required to the LCOG drawings to more accurately match field conditions. SewerCAD's interface with AutoCAD allowed for the creation of a map of the pipe layout based on the positions of the sewer lines in the LCOG map. The AutoCAD interface was also used to determine lengths and areas required for various model calculations.

3.2.4 - Phase 3: Assembling the Model

Once the layout is adequately represented, the physical measurements from Phase 1 are applied. In SewerCAD, this is an automatic process called 'synchronizing.' For example, when a pipe is first laid out in SewerCAD, it assumes that it is PVC, 12 inches in diameter, has no name, has flow line elevations of zero feet, and has a slope of zero. Synchronizing assigns each pipe in the layout its name, elevation, slope, size and material, based on what is entered into the spreadsheet. Almost all system element measurements, including gravity and pressure elements, were input into the model in this way.

3.2.5 - Phase 4: Establishing Loading Data

Model loading data for the collection system was established by analyzing population data and 2008 wet weather flow data. Flow data was gathered from field sensors in February 2008, and treatment plant flow recorded in January 2008.

Population

The established residential occupancy according to the City's master plan is 2.89 people per lot, with recent census figures estimating 2.85 people per household. We used a value of 4,350 as the existing service population. This is conservative, since the total sewered population was determined to be 4,300 by the city at the end of 2007.

Sewer Sheds

Tax-lots shown in the Veneta infrastructure maps were counted and grouped into seven basins or 'sewersheds.' Figure 3.1 shows the boundaries of the sewersheds.

Flow Monitoring

In January and February 2008, flow sensors were installed at three locations within the Veneta sewer network. These flow sensors measured flow from the following selected sewer areas:

- The downtown area between the railroad tracks and Hunter Avenue and between 4th and 7th Streets (see Shed 2 in Figure 3.1, Subshed 2 in map in Appendix B.1)
- The Southwest hills between Parkside Avenue and the Southern edge of Bowling Greens Estates, and between 10th and 8th Streets (see Shed 1 in Figure 3.1, Subshed 2 in map in Appendix B.1)
- The area serviced via Territorial Road South of the first manhole, and South of Hunter Avenue (see Shed 4 in Figure 3.1 or in map in Appendix B.1)

The flow sensors monitored flow velocity and wastewater depth. Accompanying software computed the total flow based on these measured values for preset time intervals. The sensors operated for three weeks in February 2008, and captured the end of a large winter storm. The flow sensor data was then arranged into minimums, peaks, and averages for the monitored time span. This data was separately tabulated for each monitored subshed. Data summaries and a discussion of the findings are provided in Appendix B.3.

It was the intent of the flow monitoring task to provide a base for calibration (see 3.2.6). As reviewed later and discussed in the Appendix, the data was not sufficient for calibration.

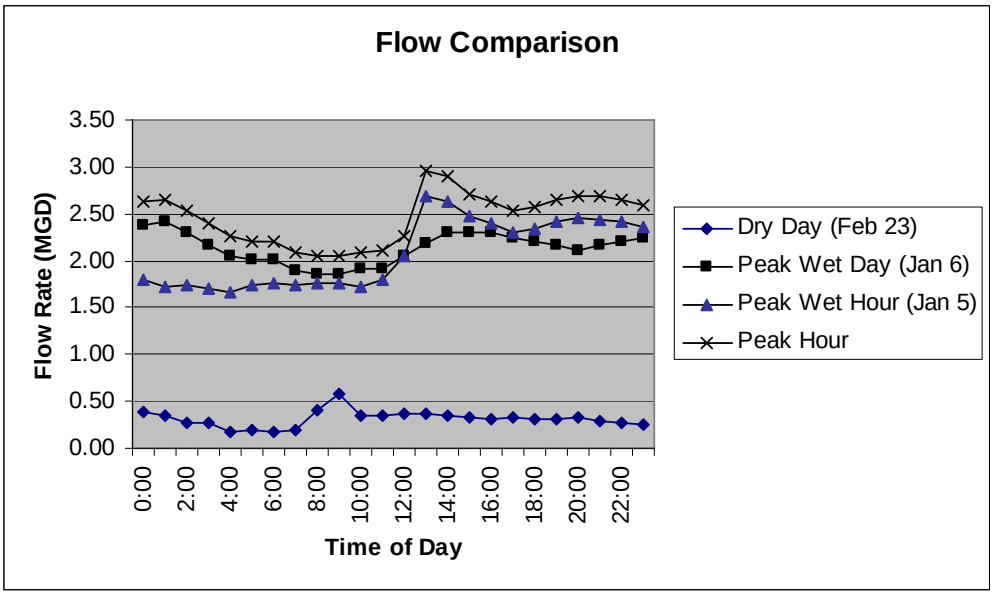
Wastewater Treatment Facility Records

The City's wastewater treatment plant monitoring documents provide the basis for determining system hydraulic loads. Measured loads for the entire city system were secured to compare these flows to the flows measured by the sensors installed in 2008. Historical flow data was compared to monitored flow data for dry weather flow computations. After data compilation, the flow patterns found at the treatment facility were used to provide a flow pattern template input for the model.

After studying hourly flow records from both treatment plant records and field sensors, flow patterns for the dry day (February 23rd), wet day (January 6th) and wettest hour (January 5th), were plotted. See Figure 3.2a. Based on the February

day, each individual in Veneta contributed 70 gallons per day to the sewer, (70 GPCPD). During the wettest hour of the wettest day recorded, each individual contributed 9.68 times that number, (680 GPCPD). At the treatment plant, the dry day flow was 300,000 gallons, and the wettest day on record was 2.2 million gallons. The flow pattern shown in Figure 3.2a for February 23rd does not show a 6:00 p.m. peak, which is normally depicted during the average work week. This is because February 23rd was a Saturday.

Figure 3.2a



Selected Storm Loading

The DEQ regulations recognize the difficulty and cost of managing and constructing wastewater treatment facilities to manage extreme wet weather conditions. Therefore the regulations require that the plant not experience failure or overflow for a five year frequency storm but if the storm is larger than a five year storm partial plant failure and/or a bypass is allowed. For this reason we have evaluated the collection system to adequately perform during a five year storm (or flow) event.

Based on the likelihood that Veneta has suffered worse storms than those recorded in the last three years (i.e. Veneta has not recorded a five-year storm at the treatment plant), a peak flow pattern (Figure 3.2b) that approximated a five-year storm was developed and used for the model. This peak flow pattern is represented as the peak hour line on Figure 3.2b. The peak hour is processed by the computer model by using the flow pattern from midnight to 1:00 p.m. as antecedent conditions to the peak hour flow that occurs at 2:00 p.m. That flow pattern yields about 2.4 million gallons per day (MGD). This design storm flow pattern represents predicted conditions during a five-year storm, but does not represent actual recorded conditions at the treatment plant. For more detailed calculations leading to the design storm, see Appendix B.4.

3.2.6- Phase 5: Model Calibration

With the model assembled and the wet weather loading established, preliminary runs of the model were conducted to calibrate the model to field conditions. This process involved the following:

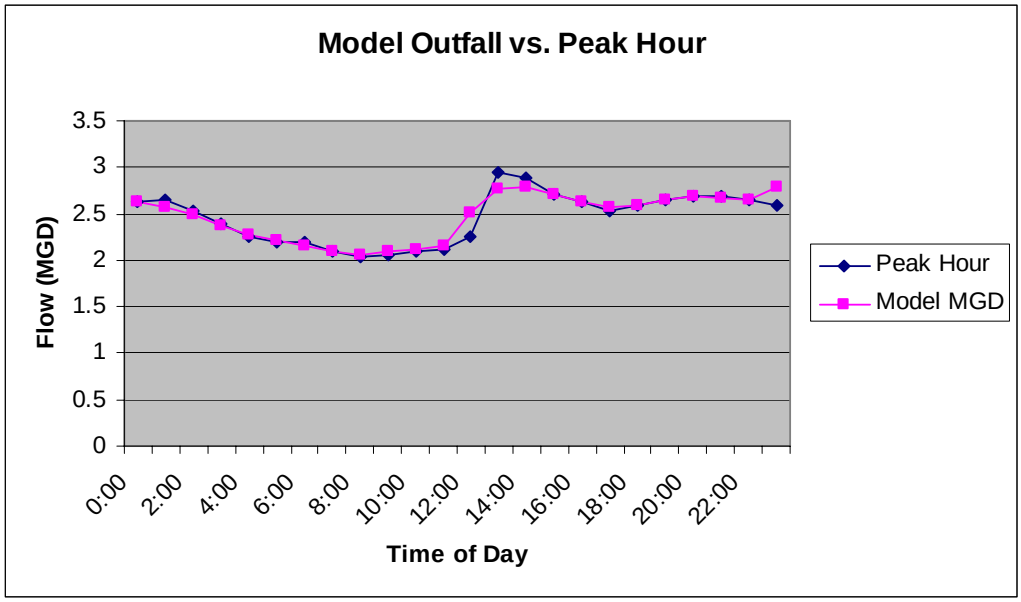
- Running the model with calculated peak hour loading pattern
- Comparing model output to known plant data, pump run data and known overflow conditions
- Adjusting loading patterns, input locations and model pump settings to match observed conditions

The model runs were repeated until a consistent correlation was found between model output and observations. The model was considered calibrated when the following criteria were met:

1. Model accurately measured flow patterns at the treatment facility
2. Demonstrate realistic pump station operation during simulations
3. Accurately identify known deficiencies in the system

As shown in Figure 3.2b, the model accurately measured flow patterns at the treatment facility.

Figure 3.2b



3.2.7- Phase 6: Model Run and Presentation

The last phase involved tabulating data from the model and creating graphical presentations of the data. These graphics are included in this chapter, as well as in Appendices B. The results of the functional, calibrated model are discussed in Section 3.4.

3.3 - Infiltration and Inflow (I/I)

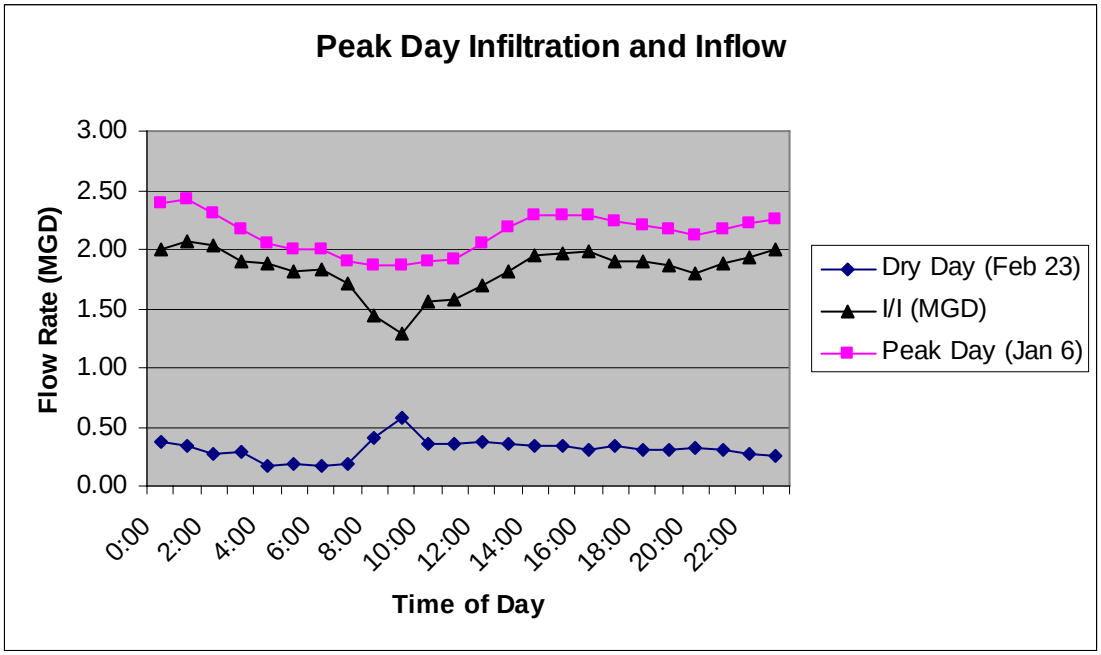
The definition of **infiltration** is: excess flow in a sewer system resulting from seepage through cracks and joints. Infiltration occurs when the water table of the soil around the pipe is at or higher than the wall of the pipe, and the pipe lacks integrity.

The definition of **inflow** is: excess flow in a sewer system resulting from direct channels of flow into the sewer. This includes runoff from precipitation, rainfall through manhole covers, or misconnected/misused laterals behind property lines. It also includes roof drains and downspouts connected to the sewer (usually inadvertently) and basement or crawl space sump pumps that connect to the sanitary sewer. Inflow occurs mainly during storms, but may also be evident during periods of high usage.

To determine infiltration and inflow for the existing collection system, flow characteristics from the peak recorded wet weather day (January 6th, 2008) and dry weather day (February 23rd, 2008) were examined. The difference estimates the total amount of flow due to infiltration and inflow.

The subtraction yields a total I/I flow for January 6th of 1.8 million gallons. The I/I flows are shown in Figure 3.3a: Figure 3.3a illustrates an odd drop in infiltration and inflow between the hours of 8 a.m. and 10 a.m. This is because the I/I line shown is a subtraction between the peak day and the low flow day of February 23rd. And because February 23rd was a Saturday, the pattern is different. The conclusion remains that peak I/I is estimated at 1.8 mgd.

Figure 3.3a



It would be helpful in any future flow analysis to have the ability to correlate sewershed groundwater, depth to flow. This could be accomplished by installation

of groundwater monitoring wells at selected sites and taking reading of groundwater depths about every two weeks during winter season. When implemented such monitoring points should be considered temporary for the purpose of obtaining groundwater level information to ensure compliance with statues relating to monitoring wells.

A significant storm on October 3, 2008 provides a unique opportunity to evaluate inflow contributions to extraneous flow. October 3 was a Friday and 0.4 inches of rainfall was experienced overnight. From Friday until the rainfall gauge reading on Saturday another 1.1 inches of rainfall occurred. As shown on Table 3.3c influent flows increased from a base of 0.147 to 0.177 MGD over the two day period - an increase of 30,000 gallons per day. By the 8th of October the flow had subsided to 0.162 MGD. An examination of the subsidence in flow would suggest a 15,000 gallon per day inflow amount. We conclude that inflow contributions to extraneous flow are minimal. An examination of the effluent flows in Table 3.3c is more difficult to interpret. Perhaps the more logical conclusion drawn from the Table is that there is considerable inconsistency between the two metering systems.

Table 3.3c

October 3, 2008 – Storm Flow Measure			
Date	**Influent-MGD**	**Effluent-MGD**	**Rainfall-Inches**
10-2-08	.147	.188	.25
10-3-08	.160	.358	.40
10-4-08	.177	.283	1.1
10-5-08	.208	.383	.02
10-6-08	.206	.373	.01
10-7-08	.175	.381	.01
10-8-08	.162	.320	0

3.4 - Current Deficiencies

The model runs identified collection system deficiencies. Pipes are earmarked as deficient if they flooded (i.e. the flow level was at or above the top of the pipe). Pipes are given a “Level Warning” if the flow in the pipe is at or above 75% of the pipe capacity. The model analysis identified nine pipe segments (see Table 3.4a) that are either experiencing level warning designations or the pipe is flooded during the peak hour storm.

Table 3.4a

Current Model Pipe Deficiencies					
Location	**Upstream Node**	**Down-stream Node**	**Total# of Pipes**	**Pipe Sizes in inches**	**Problem**
Along driveway to WWTP	MH Meter	WWTP	6	21, 27	Pipe flooded
Between Parkside and Sertic	MH 8-A	MH I-16	1	8	Pipe flooded
Along Sertic to 7th to Hunter	MH I-9	MH I-14	5	15, 18	Level warning
Hunter from 6th to Pine	MH 246	MH I-10	10	8, 15, 21	Pipe flooded
Hunter	MH 247	MH 246	1	21	Level warning
3rd between McCutcheon and Hunter	MH F-1	MH I-7	1	8	Level warning
Territorial from Broadway to Hunter	MH I-2	MH I-4	3	12	Pipe flooded
Territorial from south of Cheney to Hunter, incl. Bolton Hill branch	MH T-3	MH I-5	10	8	Pipe flooded
West branch off Territorial between McCutcheon and Hunter	MH G-1	MH I-3a	1	8	Pipe flooded

Table 3.4b identifies four manholes predicted to overflow into the street during Peak Day conditions. Figure 3.1 shows the locations of the flooding points. Additional details can be found in Appendix B.1.

Table 3.4b

Current Model Manhole Deficiencies			
Location	**Manhole Name**	**Problem**	**Observed?**
Territorial between Bolton Hill and Hunter	MH Ba-3	Overflow	No
Territorial between Bolton Hill and Hunter	MH Ba-F	Overflow	No
Bolton Hill branch East of Territorial	MH T-D	Overflow	Yes
Treatment facility driveway	MH 301	Overflow	Yes

Two of the above locations have not been observed to flood, but do flood under modeled design storm conditions. Further discussion is found in Appendix B.5.

The flooded elements indicated in the pipes and manholes are scheduled for immediate resolution in the Capital Improvement Plan (Chapter 7). Where pipes

are experiencing level warning conditions the improvements are scheduled in future budget cycles.

3.5 – Projected Deficiencies

The first stage in finding a workable 2030 collection system upgrade is analysis of how the existing collection system reacts to 2030 population loads. This identifies and locates which sections of the sewer must be upgraded to provide for 2030 loads.

Veneta's projected service population for 2030 is 9,960, as discussed in Chapter 2. To assess 2030 performance, the model was run with existing physical parameters and identical winter load patterns, but with a projected 2030 design flow rate. The future design flow model run identified the following:

- 26 manholes in the system were predicted to flood
- Most trunk lines in the gravity network were predicted to exceed capacity
- The design flow exceeded the rated pump capacity for both West Lane and Pine Street pump stations, and at peak hour, the Pine Street wet well overflowed

A graphical representation of this situation can be found in Appendix B.6.

3.6 - Alternative Solutions

Alternative Design Overview and Definitions

This section presents a review of five alternatives for the capital improvement plan that will correct the projected deficiencies identified by the 2030 model run. Alternative 1 relies primarily on pressure system improvements and adds two pump stations. Alternative 2 relies primarily on gravity pipe, upsizing on Hunter and the installation of 21-inch gravity line through the Shadow Ridge subdivision, resulting in the addition of only one pump station. Alternative 3 also relies primarily on gravity pipe upsizing, but uses several gravity pipe bypasses in an attempt to reduce construction on Hunter and Territorial. This alternative also requires the addition of only one pump station. Alternatives 4 and 5, like Alternative 3, rely primarily on gravity pipe upsizing, and attempt to reduce construction on Hunter and Territorial. Alternative 4 accomplishes this using a new gravity pipe along McCutcheon, and routing all pressure flows on Hunter through the Public Works yard. Alternative 5 is the same as Alternative 4; however, construction costs are minimized by routing all existing flows from Hunter and Territorial to the new McCutcheon sewer pipe.

A separate study was required for the area within the UGB north of Jean's Road and east of Territorial Highway that is not currently served. The study produced projects that influenced the development of the three main alternatives. The complete North East Area Study is presented in Appendix B.7.

* A glossary is provided in section 3.10 of this chapter to clarify common terms used in the planning, design and construction of sanitary sewer systems.

3.6.1 - Common Projects

This section reviews projects common to each design alternative. Presenting them here prevents repetition in the discussion of each alternative.

- ***Manhole Chimney at Treatment Plant*** – Manhole 301, next to the wastewater treatment facility driveway, floods with all alternatives. The addition of a one foot manhole rim extension will prevent flooding in this manhole during peak storm events.
- ***8th Street Gravity Branch*** – The eight-inch gravity pipe under Parkside and under 8th Street in the Bowling Green development will be upsized to 10 or 12-inch PVC pipe to meet 2030 peak load requirements. These pipes are shown on the Alternative maps (Figures 3.5, 3.6, 3.7 and 3.8) between MH 8-A and MH 8-B, and between MH 8-F and MH 8-I.
- ***Oak Island Gravity Branch*** – Gravity pipe under Cheney and under Oak Island will be upsized to 10-inch PVC pipe to meet 2030 peak load requirements. These pipes are shown on the Alternative maps (Figures 3.5, 3.6, 3.7 and 3.8) between MH 502 and MH 503, and between MH D-1 and MH D-3.
- ***New East Pump Station*** – The Eastern sections of Veneta will require a pump station to service lower elevations.

3.6.2 - Alternative 1

Concept Description

Alternative 1 relies on a new pressure network to serve future developments within most of Veneta's urban growth boundary. The pressure network would bypass much of the existing gravity system by re-routing all Pine Street, West Lane, and two new pump stations around the central city core and directly to the wastewater treatment facility. This is the only alternative that would not require the addition of a new influent pump assembly to the wastewater treatment facility. This alternative requires two new pump stations, one on Huston and one in the Northeast section of Veneta. This alternative also calls for capacity upgrades for Pine Street and West Lane pump stations. The northeast section of Veneta will be serviced by individual pump stations wherever the topography of the area prohibits gravity service (See Appendix B.7 for more details). A graphic depiction of the Alternative 1 sewer plan is shown in Figure 3.5.

Construction Elements

Alternative 1 requires the following construction elements:

- 2,290 feet of eight-inch concrete piping under Territorial Highway must be abandoned and/or replaced with 10, 12 or 15-inch polyvinyl chloride (PVC) gravity pipe. Some of these sections are candidates for pipe bursting. Whether the difference is constructed by conventional trench and pipe, or pipe bursting can be determined during design.
- 70 service laterals would need to be re-connected to new sewer pipe

- 12 new manholes would need to be installed with new sewer pipe
- 9,290 feet of new 12-inch pressure pipe constructed from the wastewater treatment facility to Hunter Road between Pine and Huston. This pressure line follows a path roughly parallel to the railroad tracks, then south to Hunter, as shown in Figure 3.5.
- Construct a new pump station north of Jean's Road, about 150 feet south of the urban growth boundary. Install duplex 10-horsepower pumps and 2,370 feet of four-inch diameter force main. The site for the station is labeled "New North Pump Station" on Figure 3.5.
- Construct a new pump station on Huston Road about 800 feet South of Hunter Avenue. Install dual 50-horsepower pumps and 4,920 feet of 8-inch diameter force main. The approximate site for the station is labeled "New East Pump Station" in Figure 3.5.
- Disconnect the force main for West Lane pump station and re-route and reconnect the discharge into the new pressure network. 500 feet of force main would be abandoned.
- Upgrade West Lane pump station with two 10-horsepower pumps
- Disconnect the force main for Pine Street pump station and re-route to the new pressure network. This requires 454 feet of 12-inch diameter extension to the existing force main to run East on Hunter Road to reach the proposed pressure network.
- Upgrade Pine Street pump station with two 100-horsepower pumps

3.6.3 - Alternative 2

Concept Description

Alternative 2 consists of considerable upsizing and re-routing of the gravity sewer on Hunter from Territorial through the Shadow Ridge subdivision. The new trunk will tie in to the system at the manhole at the gate of the wastewater treatment plant. Alternative 2 calls for one new pump station on Huston (same design location as Huston pump station specified in Alternative 1) and a new screw pump or upgrade at the wastewater treatment facility. The northeast section of Veneta will be serviced by individual pump stations wherever the topography of the area prohibits gravity service (See Appendix B.7 for more details). A graphic of Alternative 2 can be found in Figure 3.6.

Construction Elements

Alternative 2 requires the following construction elements:

- Upsize 1,550 feet of eight-inch polyvinyl chloride (PVC) pipe along Hunter in the Shadow Ridge development to 21 inch PVC pipe. This reaches from MH 303 to MH N7 as shown in Figure 3.6.

- Construct 680 feet of new 21-inch PVC pipe to connect the main sewer line along Hunter to the sewer line in the Shadow Ridge development. This reaches from MH N7 to MH N10 as shown in Figure 3.6.
- Upsize 1,770 feet of 15 and 18-inch concrete pipe to 21-inch PVC gravity pipe under Hunter Road. Some of these sections are candidates for pipe bursting. This reaches from MH I-4 to MH I-11 as shown in Figure 3.6.
- Upsize 2,430 feet of eight and 12-inch concrete pipe to 10, 12, 15, or 21-inch PVC gravity pipe under Territorial Highway. Some of these sections are candidates for pipe bursting. This reaches from MH I-3a to MH Ba-6 as shown in Figure 3.6.
- 135 service laterals would need to be re-connected to new sewer pipe.
- 29 new manholes would need to be installed along with new sewer pipe.
- Construct a new pump station on Huston Road about 800 feet South of Hunter Avenue. This station requires dual 30-horsepower pumps and 4,920 feet of eight-inch diameter force main. The site for the station is labeled "New East Pump Station" in Figure 3.6.
- Upgrade West Lane and Pine Street pump station's pumps. West Lane station will require two 15-horsepower pumps, and Pine Street station will require two 40-horsepower pumps.

3.6.4 - Alternative 3

Concept Description

Alternative 3 focuses on a gravity pipe solution, and explores two major, and several minor re-routes to the trunk line of the sewer network. Alternative 3 was developed to find ways of avoiding major construction on Hunter and Territorial Highway, to minimize disruption on major roads, and therefore reduce costs. This would be a major advantage over Alternative 2, which relies heavily on construction in major roads. Alternative 3 requires considerable resizing of gravity pipes and a reroute of a pressure line to provide capacity to accommodate future needs. As Figure 3.7 shows, Alternative 3 fails to achieve this goal because it still requires construction of a major sewer line in Hunter Road. One pump station needs to be built in this alternative, along with a pump upgrade at the wastewater treatment facility. The northeast section of Veneta will be serviced by individual pump stations wherever the topography of the area prohibits gravity service (See Appendix B.7 for more details). A graphic of the sewer plan for Alternative 3 is provided in Figure 3.7.

Construction Elements

Alternative 3 requires the following construction elements:

- Upsize 1,550 feet of eight-inch polyvinyl chloride (PVC) pipe along Hunter in the Shadow Ridge development to 21-inch PVC pipe. This reaches from MH 303 to MH N7 as shown in Figure 3.7.

- Construct 680 feet of new 21-inch PVC pipe to connect the main sewer line along Hunter to the sewer line in the Shadow Ridge development. This is shown as Bypass 1 in Figure 3.7.
- Upsize 1,160 feet of 15-inch concrete pipe to 18-inch PVC gravity pipe under Hunter Road. Some of these sections are candidates for pipe bursting. This reaches from MH I-4 to MH I-9 as shown in Figure 3.7.
- Construct 2,110 feet of new 12 and 15-inch PVC pipe to connect the sewer line along Territorial to the sewer line under Sertic. This is shown as Bypass 4 in Figure 3.7.
- Upsize 1,270 feet of eight-inch concrete pipe to 10-inch PVC pipe under Sixth Street north of Hunter. This reaches from MH H2-6 to MH H2-1 as shown in Figure 3.7.
- Construct 660 feet of new 10-inch PVC gravity pipe to connect the sewer line on Sixth north of Hunter to the sewer line on Sixth and Woodland. This line will cross Hunter without any connections. This is shown as Bypass 2 in Figure 3.7.
- Construct 90 feet of new eight-inch PVC gravity pipe to connect the sewer line on Sixth Street south of Hunter to the sewer line on Sixth and Woodland. This reaches from MH 6-B to MH N-2 as shown in Figure 3.7.
- Upsize 1,540 feet of eight-inch concrete pipe to 10 or 18-inch PVC gravity pipe under Territorial Highway. Some of these sections are candidates for pipe bursting. This reaches from MH I-4 to MH I-5, and MH Ba-F to MH Ba-6, as shown in Figure 3.7.
- Construct 1,237 feet of new six-inch PVC pressure pipe along Waldo to connect West Lane force main to manhole at 5th and Broadway. This is shown as Bypass 3 in Figure 3.7.
- 180 service laterals would need to be re-connected to new sewer pipe.
- 39 new manholes would need to be installed along with new sewer pipe.
- Construct a new pump station on Huston Road about 800 feet South of Hunter Avenue. Install dual 30-horsepower pumps and 4,920 feet of eight-inch diameter force main. The site for the station is labeled “New East Pump Station” in Figure 3.7.
- Upgrade West Lane and Pine Street pump stations’ pumps. West Lane station will require 15-horsepower pumps, and Pine Street station will require 40-horsepower pumps.

3.6.5 – Alternative 4

Concept Description

Alternative 4 focuses on a gravity pipe solution, and explores a major re-route to the trunk of the sewer network. Alternative 4, like Alternative 3, was developed to avoid large amounts of construction on Hunter and Territorial. Unlike Alternative 3,

Alternative 4 finds a way to expand the gravity network away from Hunter and Territorial. Pressure lines from Pine Street and New East pump stations would be routed North to the City Public Works Yard at the East end of Broadway, where it would enter an expanded gravity network running West down Broadway. The expanded line would turn South at 2nd into a new 21-inch gravity bypass, which turns West again at McCutcheon and follows City streets the rest of the way to the treatment facility. One pump station needs to be built in this alternative, along with a new screw pump at the wastewater treatment facility. The northeast section of Veneta will be serviced by individual pump stations wherever the topography of the area prohibits gravity service.

Construction Elements

Because the construction elements for Alternative 4 are cost prohibitive, and are nearly identical to Alternative 5, they have been omitted from this section. Please see section 3.6.6 – Alternative 5 below for an economical construction plan.

3.6.6 – Alternative 5

Concept Description

Alternative 5 focuses on gravity pipe solution, and explores a major re-route to the trunk of the sewer network. Alternative 5 is similar to Alternative 4, except it requires much less construction. Alternative 5 does not require pressure lines from Pine Street and New East pump stations connected to Hunter. However, all flows from Hunter East of Territorial are routed north to the 21-inch bypass on McCutcheon. All flows from Territorial South of Hunter are routed north to McCutcheon as well. One pump station needs to be built in this alternative, along with a new screw pump at the wastewater treatment facility. The northeast section of Veneta will be serviced by individual pump stations wherever the topography of the area prohibits gravity service (See Appendix B.7 for more details). A graphic of the sewer plan for Alternative 5 is provided in Figure 3.8.

Construction Elements

Alternative 5 requires the following construction elements:

- Construct 4,310 feet of new 21 and 24-inch PVC sewer pipe along McCutcheon from Territorial to 8th, North along 8th to Dunham, then West along Dunham through undeveloped areas to the Treatment Facility property. This reaches from MH 303 to MH I-3 as shown in Figure 3.8.
- Tie in existing pipes to new 21-inch pipe on McCutcheon, including pipes on Territorial, 3rd, 4th, 5th, 6th and 7th. 50 feet south of each tie-in, install a clean-out and abandon pipe north of clean-out, to the new construction.
- When need arises, construct 440 feet of 21-inch PVC sewer pipe along Territorial from the Eastern junction node at Territorial and Hunter to the new 21-inch PVC sewer pipe on McCutcheon. This will link all flows South from Territorial and East from Hunter and Pine Streets to the new line on McCutcheon and off of the existing line on Hunter. This reaches from MH I-3 to MH I-5 as shown in Figure 3.8.

- Upsize 2,230 feet of 8-inch concrete pipe to 10, 12, or 15-inch PVC gravity pipe under Territorial Highway. Some of these sections are candidates for pipe bursting. This reaches from MH I-5 to MH Ba-6 as shown in Figure 3.8.
- 95 service laterals would need to be re-connected to new sewer pipe.
- 25 new manholes would need to be installed along with new sewer pipe.
- Construct a new pump station on Huston Road about 800 feet South of Hunter Avenue. This station requires dual 30-horsepower pumps and 4,920 feet of 8-inch diameter force main. The site for the station is labeled "New East Pump Station" in Figure 3.8.
- Upgrade West Lane and Pine Street pump station's pumps. West Lane station will require two 15-horsepower pumps, and Pine Street station will require two 40-horsepower pumps.

3.7 –Alternatives Analysis

This section analyzes the four collection system improvement alternatives outlined above (Alternatives 1, 2, 3 and 5). Cost estimates are provided in Section 3.8. The alternatives analysis compares and contrasts various features associated with the planning, construction and operation of each alternative. For convenience, the features are broadly categorized under the headings "Environmental" and "Social":

- Environmental considerations include environmental risks to natural and urban environments, energy demands, long-term service needs and reliability.
- Social/cultural considerations include safety, street and traffic disruption, utilities disruption, land acquisition and easements.

Environmental Concerns

Environmental Risks

Environmental risks show the measure of risk and/or extent of environmental impact on a region during construction, or operation of an alternative. This includes risk/extent of wetland disruption, natural habitat disruption, air pollution, noise pollution and contamination from raw sewage.

Energy Usage

Energy usage is the measure of energy used to operate an alternative system. The most significant energy use is the electricity used to power pump stations. This does not include the fuel used by City employees to monitor the pump stations or other parts of the collection system, or the energy used at the treatment plant.

Service Requirements and Reliability

Service requirements and reliability involves the qualification of resources used to monitor, maintain and repair the collection system. This includes measuring the resources used and staffing needed to clean or unclog pipes and pumps, repair damage, replace parts and implement system failure

contingencies (i.e. managing overflow due to inoperable pumps or clogged pipes).

Social Concerns

Construction

Social assessment of construction issues involves as assessment of public safety concerns, disruption of street usage, and disruption of public utility service. Construction concerns will depend largely on the depth of trenches being constructed and the time required for construction. Safety issues include relative safety of construction workers, City maintenance workers, residents and by-standers. Street disruption issues include how often streets are narrowed or closed, how much pedestrian traffic is affected, and how often vehicle access is impaired for residences or businesses. Utility disruption issues include how often power, water, or sewer may be unavailable.

Land Acquisition and Easements

Land Acquisition and easements looks at the measure of how much land the City must lease or acquire from private land owners to be able to implement a collection system alternative.

Table 3.7 on the next page represents a summary of collection system upgrade alternatives. Key features of each alternative are listed side by side for easy comparison.

Table 3.7

Alternative Comparison Summary

Alternatives	Environmental Impact	Energy Usage	Service Requirements	Construction Issues	Land Acquisition & Easements
1	Some wetlands impact	Highest usage	Requires most service	Safest for workers, least street & potential utility disruption	Most land & easements
2	No wetlands impact	Lowest usage	Requires some service	Some safety concerns, some street disruption, some utility disruption	Some land & least easements
3	No wetlands impact	Some usage	Requires some service	Some safety concerns, most street disruption, most utility disruption	Least land & easements
5	Less wetlands impact	Lowest usage	Requires some service	More safety concerns, some street disruption, some utility disruption	Some land & easements

Alternative 1

Environmental

Environmental impact – Alternative 1 requires construction of approximately 1,300 feet of pipe in identified wetlands. It requires construction adjacent to approximately 9,800 feet of pipe in or around residences and businesses. It requires the construction of two new pump stations, one of which is in a residential area. This will cause wetland disruption, and temporary noise and air pollution typically associated with construction activities.

Power usage – Alternative 1 requires the operation of four pump stations. The total estimated electricity usage for these pumps between 2007 and 2030 is 912,100 kWh. This is approximately 10 percent more energy than the gravity alternatives.

Service requirements/reliability – Alternative 1 requires maintenance on four pump stations and 16,530 feet of pressure pipe in addition to the existing gravity system. Typically a sanitary sewage pump station will require one-half to one hour per day of monitoring and periodic routine maintenance or minor repairs by Public Works personnel. Because this alternative has one more pump station, the collection system will require more time by Public Works personnel than any other alternatives.

Social

Construction – Alternative 1 requires some trenches be up to 16 feet deep. Ninety percent of trenches will be between 4 and 12 feet deep. Construction contractors will be required to follow safety procedures associated with sewer construction. Safety issues with respect to equipment maintenance include entry into confined spaces such as into manholes. The City has safety procedures in place for entry into confined spaces. The additional high voltage power sources in pump stations present another source of risk. Alternative 1 requires construction to approximately 2,300 feet of pipe along major collectors, and approximately 1,500 feet of pipe along minor collectors or local streets. This will cause temporary disruptions to local businesses, including disruption of sewer, water or power service. There will be fewer instances of street disruption in this alternative than with the other alternatives. Few if any complete road closures will be required. There will be little impact to pedestrian traffic.

Land acquisition and easements - Alternative 1 requires 9,000 square feet of land acquisition for pump stations. The force main would require 1,680 lineal feet of 10-foot wide easements. This would involve negotiations with at least five, possibly more land owners in Veneta.

Alternative 2

Environmental

Environmental impact - Alternative 2 requires no construction of pipe in identified wetlands. It requires construction of 14,100 feet of pipe around residences and businesses. It requires the construction of one new pump station in a residential

area. This will cause some temporary noise and air pollution typical of construction activities.

Power usage – Alternative 2 requires the operation of three pump stations. The total estimated electricity usage for these pumps between 2007 and 2030 is 825,800 kWh.

Service requirements/reliability – Alternative 2 requires maintenance on three pump stations and 4,920 feet of pressure pipe in addition to the gravity system. This alternative requires one fewer pump station to service and maintain than the first alternative. Each pump station will require one-half to one hour per day of monitoring and periodic routine maintenance or minor repairs by Public Works personnel.

Social

Construction – Alternative 2 requires some trenches to be 22 feet deep. Sixty percent of trenches will be between 4 and 12 feet deep. Construction contractors will be required to follow typical safety procedures associated with sewer construction. Safety issues with respect to equipment maintenance include entry into confined spaces such as into manholes. The City has safety procedures in place for entry into confined spaces. The additional high voltage power sources in pump stations present another source of risk. Alternative 2 requires construction to approximately 5,750 feet of pipe along major collectors, and approximately 4,970 feet of pipe along minor collectors and local streets in the City. This will cause some temporary disruptions to local businesses, including disruption of sewer, water or power service. There will be more instances of street disruption in this alternative than with the first alternative, but fewer than with the third alternative. Few if any complete road closures will be required. There will be little impact to pedestrian traffic

Land acquisition and easements - Alternative 2 requires 6,000 square feet of land acquisition for pump stations. A gravity bypass would require 682 lineal feet of 10-foot wide easements. This would involve negotiations with at least three, possibly more land owners in Veneta.

Alternative 3

Environmental

Environmental impact - Alternative 3 requires no construction of pipe in identified wetlands. It requires construction of 18,000 feet of pipe around residences and businesses. It requires the construction of one new pump station in a residential area. This will cause some temporary noise and air pollution typical of construction activities.

Power usage – Alternative 3 requires the operation of three pump stations. The total estimated electricity usage for these pumps between 2007 and 2030 is 890,700 kWh.

Service requirements/reliability – Alternative 3 requires maintenance on three pump stations and 6,160 feet of pressure pipe in addition to the gravity system. Each

pump station will require one-half to one hour per day of monitoring and periodic routine maintenance or minor repairs by Public Works personnel.

Social

Construction – Alternative 3 requires some trenches to be 22 feet deep. Fifty percent of trenches will be between 4 and 12 feet deep. Construction contractors will be required to follow typical safety procedures associated with sewer construction. Safety issues with respect to equipment maintenance include entry into confined spaces such as into manholes. The City has safety procedures in place for entry into confined spaces. The additional high voltage power sources in pump stations present another source of risk. Alternative 3 requires construction to approximately 5,750 feet of pipe along major collector streets, and approximately 12,250 feet of pipe along minor collectors or local streets in the City. This will cause some temporary disruptions to local businesses, including disruption of sewer, water or power service. There will be more instances of street disruption in this alternative than with any other alternatives. Few if any complete road closures will be required. There will be little impact to pedestrian traffic

Land acquisition and easements - Alternative 3 requires 5,020 square feet of land acquisition for pump stations. A gravity bypass would require 682 lineal feet of 10-foot wide easements. This would involve negotiations with at least three, possibly more land owners in Veneta.

Alternative 4

Alternative 4 was not assessed for environmental or social impacts, due to its similarity to Alternative 5 and its high capital costs.

Alternative 5

Environmental

Environmental impact - Alternative 5 requires construction of approximately 830 feet of pipe in identified wetlands. It requires construction of 8,500 feet of pipe around residences and businesses. It requires the construction of one new pump station in a residential area. This will cause some temporary noise and air pollution typical of construction activities.

Power usage – Alternative 5 requires the operation of three pump stations. The total estimated electricity usage matches Alternative 2.

Service requirements/reliability – Alternative 5 requires maintenance on three pump stations and 4,920 feet of pressure pipe in addition to the gravity system. Each pump station will require one-half to one hour per day of monitoring and periodic routine maintenance or minor repairs by Public Works personnel.

Social

Construction – Alternative 5 requires some trenches to be 26 feet deep. 55 percent of trenches will be between 4 and 12 feet deep. Construction contractors will be required to follow typical safety procedures associated with sewer construction. Safety issues with respect to equipment maintenance include entry into confined

spaces such as into manholes. The City has safety procedures in place for entry into confined spaces. High voltage power sources in pump stations present added risk. Alternative 5 requires construction to approximately 2,500 feet of pipe along major collectors, and approximately 7,000 feet of pipe along minor collectors or local streets in the City. This will cause some temporary disruptions to local businesses, including disruption of sewer, water or power service. There will be more instances of street disruption in this alternative than with the first alternative, but fewer than with the third alternative. Few if any complete road closures will be required. There will be little impact to pedestrian traffic.

Land acquisition and easements - Alternative 5 requires 6,000 square feet of land acquisition for pump stations. Some gravity pipe construction would require 830 lineal feet of 14-foot wide easements. This would involve negotiations with at least two, possibly more land owners in Veneta. Since these easements are in identified wetlands, a permit would be required to begin construction.

3.8 – Cost Estimates of Alternatives

Alternatives are compared both on a construction cost basis and an operating cost basis.

3.8.1 – Construction Costs

Construction costs estimates for comparison of alternatives were developed to reflect costs in the second quarter of 2008 for rural Lane County. Four sources were used to develop the estimates:

1. Recent bid sheets for work in Lane County with emphasis on a 1.7 million dollar project involving sewer and road construction for the Veneta/Lane County Bolton Hill Road project.
2. ODOT's posting and web site that tabulates ODOT bids from around the state.
3. 2008 National Construction Estimator. 56TH Edition. Craftsman Book Company. Carlsbad, Ca. and
4. BNi Building News Public Works 2009 Costbook. 16th Edition. BNiBuilding News. 2008.

Where differences existed between the sources more reliance was put on the local or regional data than the national estimators (books). The books were used more for determining costs trends and for specialty items (such as fittings) which would not be included in local bids or data bases.

A scrutiny of the costs estimates that follow will reveal that consistent prices are not tabulated for pipeline costs. For instance, in one area 8 inch sanitary sewer may cost $88 per foot and, in another area, the same pipe may cost as much as $108 per foot. This is because the various alternatives have differing depths of excavation, access, pavement structures, alignment and infrastructure complications. To accommodate these variations in construction we developed a spreadsheet that integrates various cost factors such as depth of excavation, shoring, pipe diameter, trench width, asphalt depth, cost of hauling, flagging and

safety, etc. A factor is given for all of these elements to arrive at a construction cost for project segments.

The costs presented in this document are a budget estimate. The American Association of Cost Engineers establishes the standard for a budget estimate. Although cost research and reviews of project requirements with various vendors represent solid estimates of current costs the project itself has not matured to the extent that project details, contingencies and specific requirements can be identified. The estimates are prepared using process flow sheets, layouts and assumed site conditions. Estimates of this type may be as much as 30 percent high to 15 percent low. Because the project has so many variables at this stage contingencies are added to allow for uncertainties that are unavoidably associated with a project in the early stages of development. This contingency provides allowance for unforeseen mechanical items, options that may be selected by the City or required as part of an agency permit, variations in final quantities, market fluctuations, economic and bidding climate. For this study we are using a 20% contingency.

The cost of final engineering services for these projects includes costs for special investigations, surveys, pre-design report, and geotechnical investigations. They also include preparation of design documents including drawings, specifications and contracts. And finally, engineering includes the preparation of operation and maintenance manuals, performance certifications, and, often, continuing start-up services. Engineering costs on a project will range from 13 percent up to over 25 percent. We have used a factor of 20 percent.

In addition to the contingency and engineering costs, projects have administrative and legal costs. Administration and legal costs include legal review of documents, legal preparation of easements and rights-of-way and administrative support including monitoring of permit requirements, contract payment schedules, bonds, finance fees, and other project requirements. We have applied a factor of 5 percent for administrative and legal costs.

3.8.2 – Operation and Maintenance Costs

Cost comparison of alternatives must include an analysis of annual costs. Annual costs are those costs associated with the activities and expenses for operation, maintenance and administration of the project once it is placed in service. Operation costs are principally energy costs but also include costs for chemicals and outside services. Maintenance costs include the labor, material, equipment needed to support the facilities for routine operating functions.

3.8.3 - Cost Comparison

Alternatives are compared by use of a procedure called the present worth analysis. A present worth analysis is an economic analysis where all of a project's costs are brought from the future and presented as a present worth. The procedure assumes that at a given discount rate all future costs are brought back to the present. In other words a present worth analysis is the amount of money required in the bank today to pay for the service life of the project. For this analysis we are assuming

that the return on money is 4 percent and that annual costs experience a 4% inflation factor.

Table 3.8 presents the present worth cost estimates for each collection system alternative. See Appendix B.8 for a breakdown of cost estimates for each alternative.

Table 3.8

Alternative Cost Comparison				
	Alternative 1	Alternative 2	Alternative 3	Alternative 5
Construction:	$2,842,200	$3,147,400	$3,340,200	$3,199,300
Contingencies (20%):	$568,400	$629,500	$668,000	$639,900
Engineering (20%):	$568,400	$629,500	$668,000	$639,900
Administrative/Legal (5%):	$142,100	$157,400	$167,000	$160,000
Total Estimated Construction Costs:	**$4,121,100**	**$4,563,800**	**$4,843,200**	**$4,639,100**
Operation/Maintenance:	$621,000	$591,700	$597,600	$591,700
Total:	**$4,742,100**	**$5,155,500**	**$5,440,800**	**$5,230,800**

3.9 – Selected Alternative

The collection system alternatives were presented to the City Council of Veneta on November 10th, 2008. Based on the information gathered for this document, and the wishes of the Council, we have chosen **Alternative 1** as the wastewater collection system recommendation for the City of Veneta. It is our opinion that this collection system provides the greatest number of benefits for the capital improvement plan:

- Lowest overall cost
- Lowest construction cost
- Least challenging to construct
- Greatest flexibility of design

3.10 – Glossary

The following definitions are used to clarify construction concepts and language used in the development of the alternatives:

Pipe Bursting

When increasing the size of a concrete sewer pipe by one nominal diameter, mechanical heads can be drawn through the pipe. The existing pipe is broken by the head, and a new plastic pipe is pulled into the void. This saves excavation of the existing pipe. This practice allows for pipe replacement without disrupting surface features, such as buildings, roads or vegetation.

Nominal Diameters

Pipe is manufactured in size increments called nominal diameters. Gravity sewer pipe is manufactured in standard sizes or nominal diameters of 8, 10, 12, or 15 inch, etc. The nominal diameter is the nearest inside diameter of the pipe. For comparison, the diameter of an 8 inch plastic sewer pipe is 7.92 inches. By 2030, Veneta's new sewer pipes will require a maximum nominal diameter of 21 inches.

Force Mains

Pump stations lift sewage from lower elevations to higher elevations, typically from one gravity segment to another. The force main is the segment of pipe that the sewage travels through after it is propelled by the pump. Force mains empty into either manholes or more pressure pipes.

Pipe Abandonment

If a pipe is bypassed or taken out of service, both ends of the pipe are physically disconnected from the surrounding sewer. All entrances into the pipe are then filled with concrete, as per the requirements of the Oregon DEQ.

Service Lateral

A service lateral is a sewer pipe that connects a residence or business to the main sewer line.

Manhole

Structure found every 100 to 400 feet along a main sewer pipe, used to access the sewer for maintenance from the surface.

Water Pump

Pump used to remove ground water from an excavation site.

Bypass Pump

Mobile pump used to temporarily re-route the flow of sewage from one manhole to another during construction.

Right of Way

Land owned by the City for City use, including installing sewer pipe or pump stations.

Easement

An arrangement the City makes with a private land owner to have the right to use their land, such as for construction or maintenance. Easements are generally less expensive than Right-of-way.

Gravity Pipe Upsizing

Sections 3.4 and 3.5, the model identify some gravity pipes in Veneta that are undersized. Most alternatives require that several adjacent pipe segments (as much as 2,000 feet) be upsized. The following construction methods were reviewed to determine the most feasible way to upsize the longer sections of gravity pipe:

- Trench and Replace: excavate down to existing pipes, remove and replace with new pipe.
- Pipe-burst concrete sections then excavate and re-connect service laterals.
- Construct new pipe in a new parallel trench, abandon old pipe, and then re-connect service laterals to new pipe.

The first construction technique is typical practice for installing new sewer lines. It is most cost effective when used in a new development that does not yet contain roads or other improvements. This method requires the use of a bypass pump when upsizing existing sewer, because the sewage must be conveyed downstream while the old sewer pipe is replaced. This is especially difficult to accomplish over long stretches of pipe, because the bypass pump may need to be excessively large. This method also imposes other service disruptions such as street access.

The second construction technique requires the excavation of a delivery pit and a receiving pit at existing manholes. A cable is drawn through the existing sewer pipe from the delivery pit to the receiving pit. Typically this will be from manhole to manhole and occasionally the pits will be set over multiple manholes. After the cable is drawn through the work section a mechanical head is attached to the cable. The mechanical head is a bursting tool that hammers and pushes the existing pipe apart. A polyethylene pipe is pulled into the void space created by the bursting head. The pipe is fusion welded in 20 or 40 feet sections to make a continuous pipe, typically one nominal diameter greater than the existing pipe. While the pipe is being pulled, a bypass pump is used to route wastewater past the work area. Service laterals are then excavated and attached to the new polyethylene pipe. The principal advantage of this technique is that there is less street disruption and less risk to other infrastructure. Occasionally this technique is less expensive than conventional excavation and pipe lying.

The third technique is identical to the first, except the existing sewer pipe remains active throughout construction of the new line. Since the new line is parallel to the old, when installation is finished, the old line is simply plugged up until the old pipe is connected to the new pipe. When existing pipe is asbestos cement the ability to abandon the pipe in place reduces environmental risk and costs of moving AC pipe.

The actual construction method used will depend on the task, and will ultimately be selected by the designer. The following assumptions were made for cost estimating purposes:

- For long pipe upsizing the third construction technique was assumed to apply. Most gravity sections requiring upsizing lie under streets with few surface features, are excessively long, and require large increases in nominal pipe sizes.
- For concrete pipe requiring smaller pipe size changes, the pipe-bursting technique is applied. Street disruption and environmental impact is minimized when pipe-bursting is used.

For short sections of pipe, generally manhole to manhole, the dig and replace technique is considered for cost evaluation. This recognizes that pipe-bursting may actually be cost prohibitive, due to multiple street disruptions over short distances.

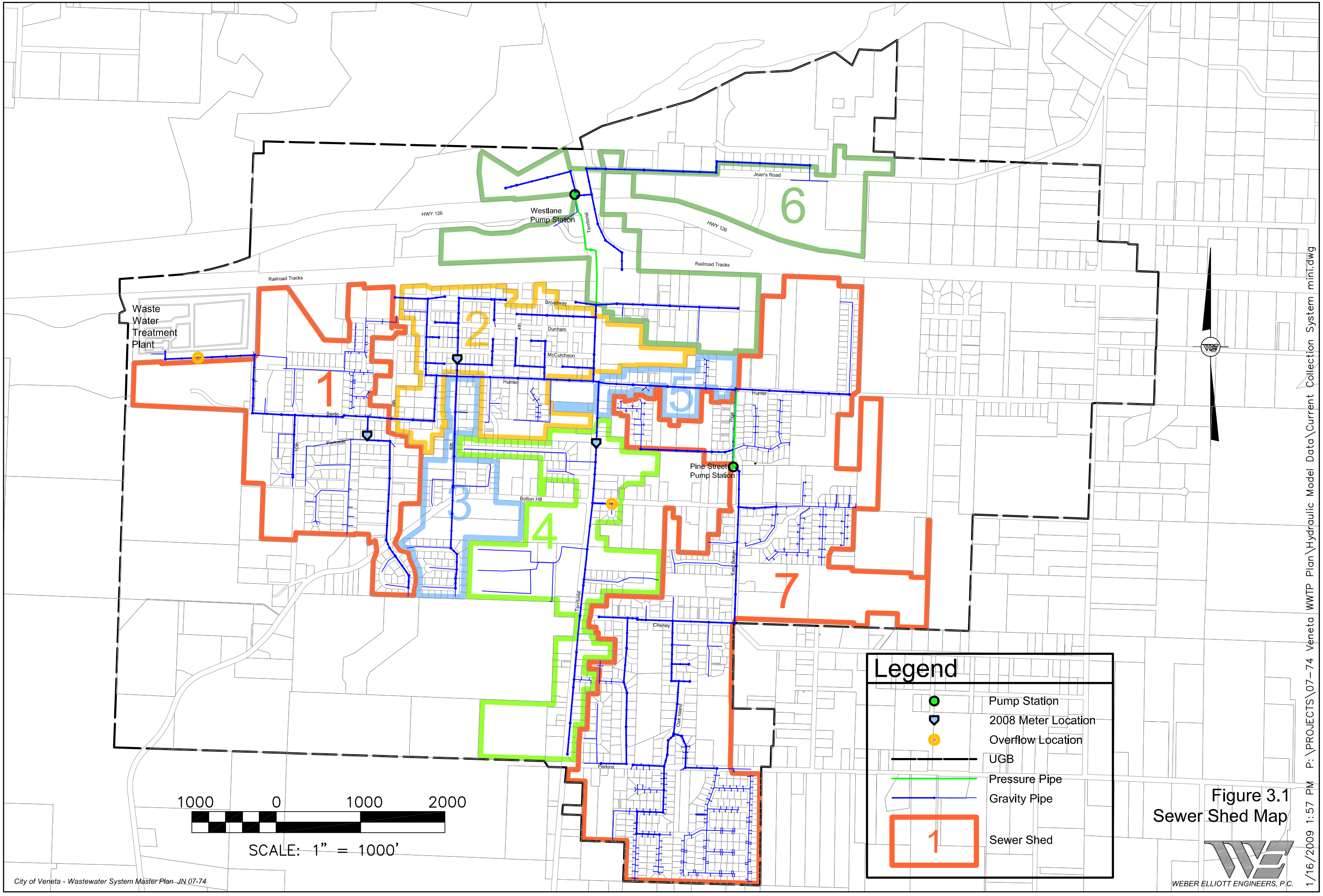


Figure 3.1
Sewer Shed Map

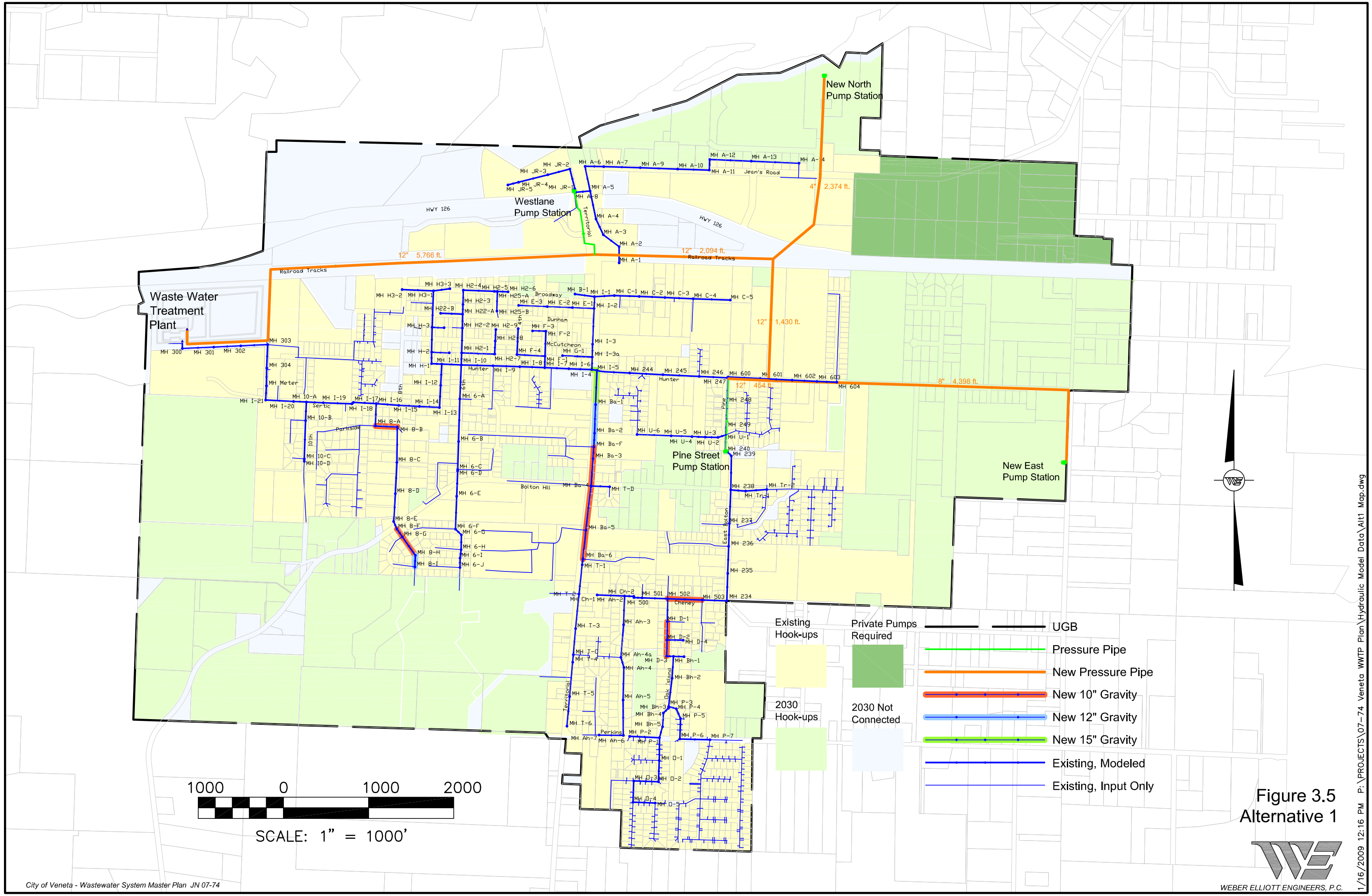


Figure 3.5
Alternative 1

WEBER ELLIOTT ENGINEERS, P.C.

City of Veneta - Wastewater System Master Plan JN 07-74

1/16/2009 12:16 PM P:\PROJECTS\07-74 Veneta WWTP Plan\Hydraulic Model Data\Alt1 Map.dwg

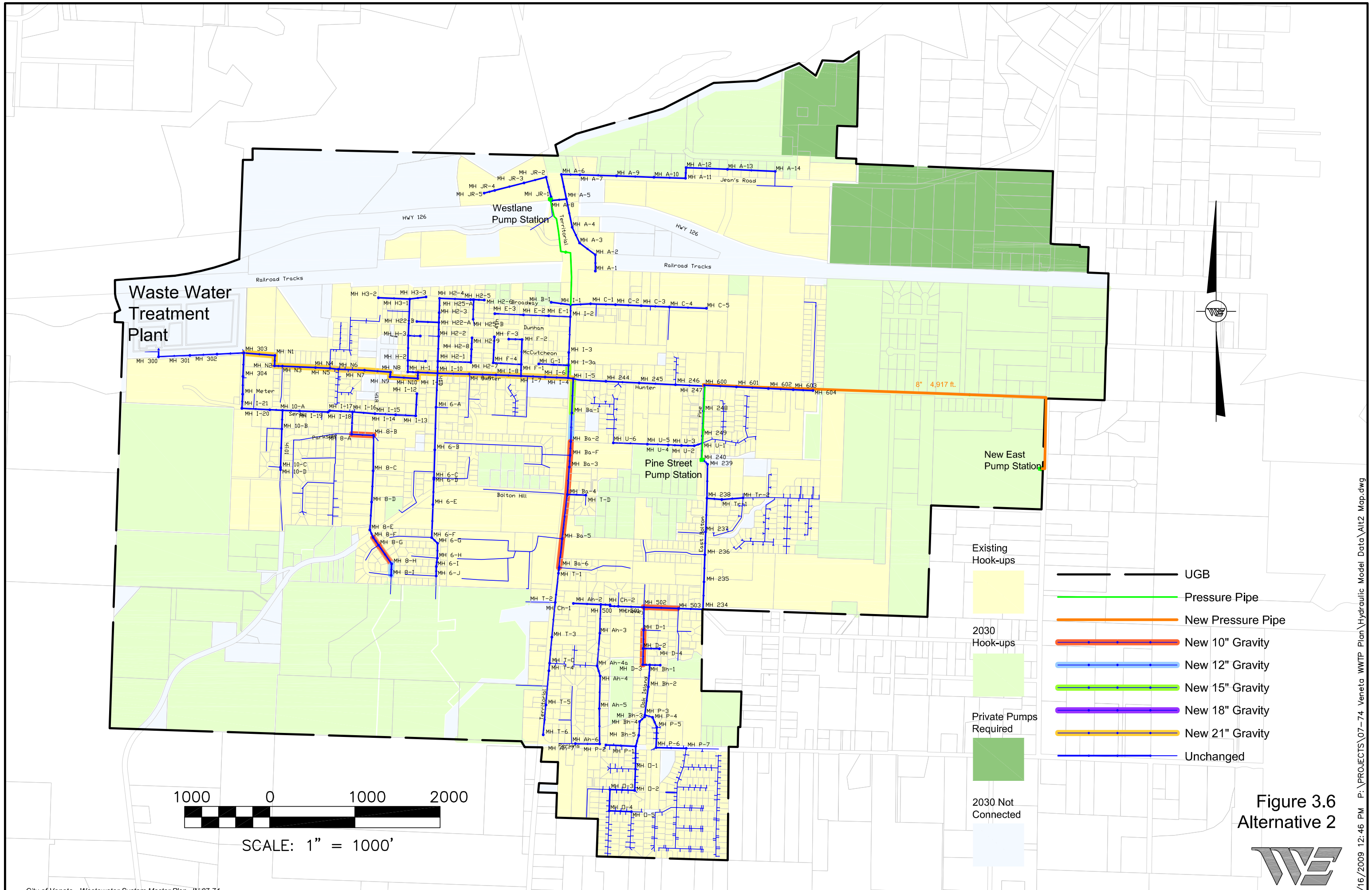


Figure 3.6
Alternative 2

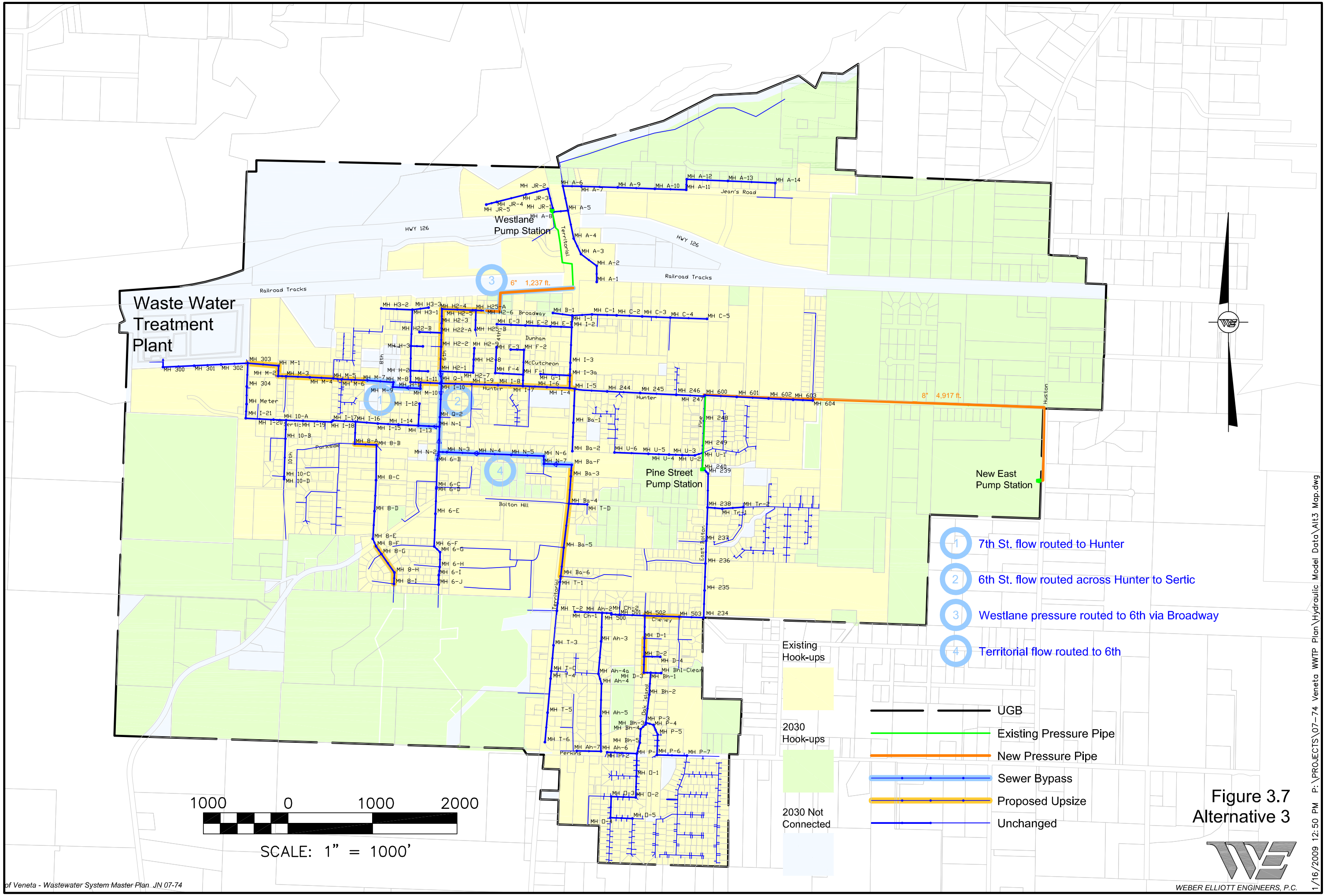
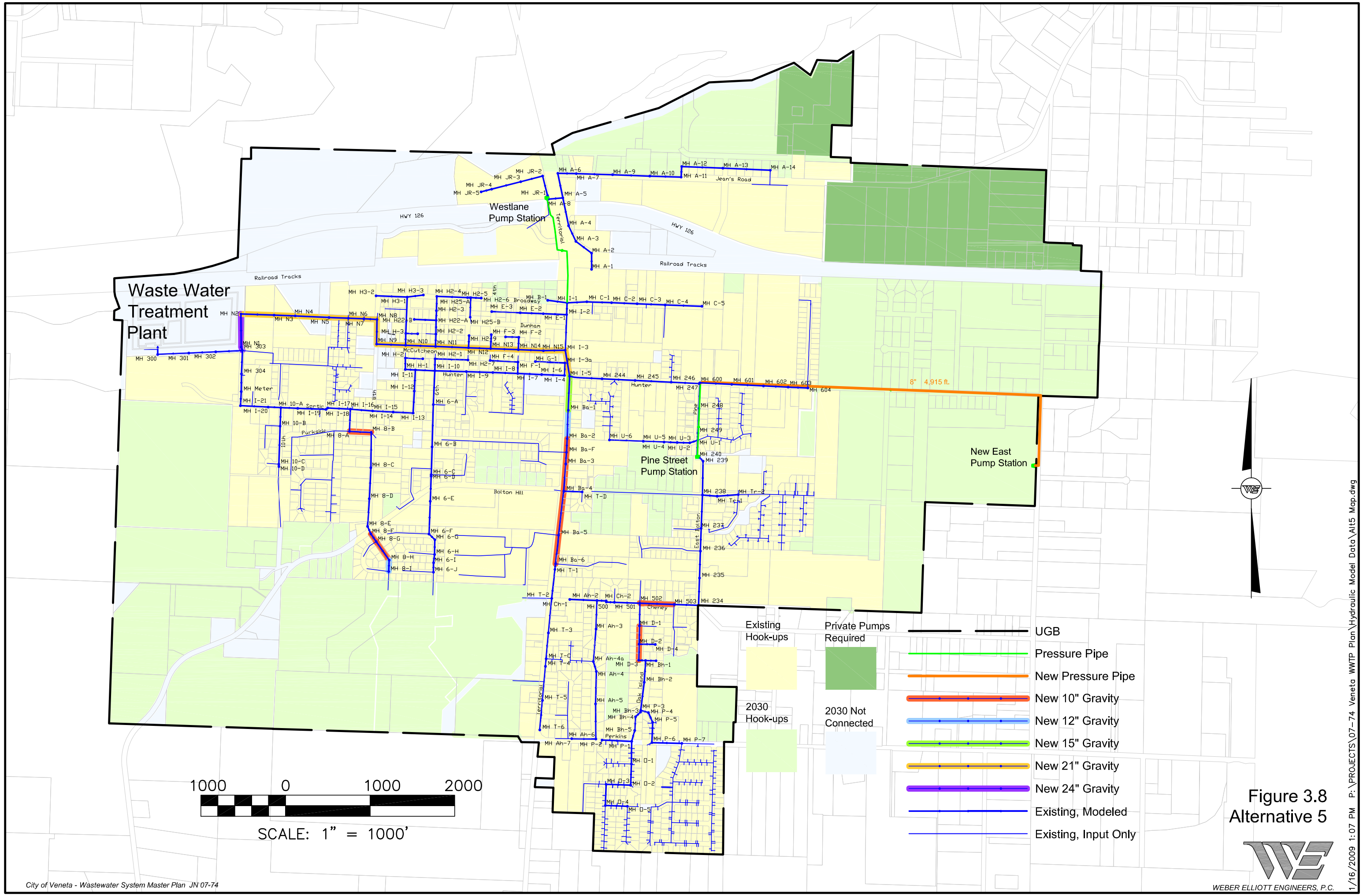


Figure 3.7
Alternative 3


Waste Water Treatment Plant
Westlane Pump Station
Pine Street Pump Station
New East Pump Station
HWY 126
Railroad Tracks
Jean's Road
Broadway
Dunham
McCutcheon
Hunter
Pine
Parkside
Bolton Hill
Cheney
Perkins
Oak Island
Territorial
East Dalton
8" 4,915 ft.
1000 0 1000 2000
SCALE: 1" = 1000'
Existing Hook-ups
Private Pumps Required
2030 Hook-ups
2030 Not Connected
UGB
Pressure Pipe
New Pressure Pipe
New 10" Gravity
New 12" Gravity
New 15" Gravity
New 21" Gravity
New 24" Gravity
Existing, Modeled
Existing, Input Only
Figure 3.8
Alternative 5
WEBER ELLIOTT ENGINEERS, P.C.
City of Veneta - Wastewater System Master Plan JN 07-74
1/16/2009 1:07 PM P:\PROJECTS\07-74 Veneta WWTP Plan\Hydraulic Model Data\Alt5 Map.dwg

4. REGULATORY CRITERIA

Chapter Summary

This chapter presents an overview of current and anticipated future Oregon Department of Environmental Quality (DEQ) regulatory criteria relative to permitting and operation of a wastewater treatment facility. These criteria establish the design guidelines for future plant upgrades, modifications or expansions. The chapter is organized into the following sections:

4.1 Regulatory Criteria

4.2 Future Regulatory Criteria

4.3 Water Reuse Regulations

Section 4.1 outlines the general regulatory requirements for the permit. It includes a review of organic and sediment load, bacterial limits, general operational requirements and management of biosolids. Section 4.2 describes future discharge and operational requirements. As future requirements will impact the plant's performance and expectations the discussion includes a review of how the existing plant may meet future requirements with an increasing population and associated sewer load growth. Section 4.3 discusses opportunities in water reuse to help remain in compliance with future discharge requirements and to provide a perspective regarding comprehensive water resource management.

The permit can be found in its entirety in Appendix D.

4.1 – Regulatory Criteria

The regulatory criteria for design and operation of a wastewater treatment facility are prescribed in the discharge permit issued by the Oregon Department of Environmental Quality (DEQ). The DEQ bases the permit limits on the ability of the resource to accept additional pollutants. A treatment plant is considered deficient when the plant fails to deliver an effluent that meets the permit criteria through a variety of inflow and operating conditions.

The permit prescribes discharge limits in four general parameters reviewed in Section 4.1.1 and 4.1.2. The areas of regulation are:

- Organic load (measured as BOD_5)
- Sediment load (measured as TSS)
- Bacterial load (measured as *E-coli* or Total Coliforms)
- General Suitability (measured as acidity and temperature)

In addition to the specific and quantitative criteria prescribed in the parameters indicated above, the permit also prescribes criteria for general chemical

suitability, good housekeeping, management and disposal of residuals and reporting. These additional criteria are reviewed in Sections 4.1.3 through 4.1.6.

4.1.1 - Organic and Sediment Load (BOD and TSS)

Tables 4.1a and 4.1b show the permit limit for BOD and TSS from Veneta's plant. Section 5, of this report, reviews the plant's success at meeting these limits.

Table 4.1a

Winter Discharge Permit Limit					
November 1-April 30					
Parameter	**Average Effluent Concentrations**		**Monthly Average lb/day**	**Weekly Average lb/day**	**Daily Maximum lbs**
	Monthly	Weekly			
BOD_5	30 mg/L	45 mg/L	130	200	260
TSS	30 mg/L	45 mg/L	130	200	260

Table 4.1b

May/October Discharge Permit Limit					
May1-31 & October 1-31					
Parameter	**Average Effluent Concentrations**		**Monthly Average lb/day**	**Weekly Average lb/day**	**Daily Maximum lbs**
	Monthly	Weekly			
BOD_5	10 mg/L	15 mg/L	44	66	88
TSS	10 mg/L	15 mg/L	44	66	88

The limits defined in Tables 4.1a and 4.1b are likely not to be amended. The discharge limits shown represent an assessment of the amount of load a municipal treatment facility can impose on the natural waterway. A plant that is processing a million gallons of waste per day with a discharge of 10 mg/L contributes an effluent load of 83 pounds. Thus, Veneta's plant processing a million gallons per day is required to produce a BOD_5 and TSS effluent of about 5.3 mg/L to meet the monthly average (May & October) discharge requirement of 44 pounds per day.

This concept of maximum daily load and effluent limits is reviewed more fully in Chapter 5. Although the City's Biolac system has managed to consistently provide a BOD_5 and TSS discharge for weeks on end at less than 5 mg/L (I.E. November 2007, December 2007, October 2007) an increase in hydraulic load exceeding 1 MGD will exceed performance expectations of the existing plant. It is a very high expectation to have an extended aeration plant to consistently provide an effluent below 10 mg/L.

The effluent values shown in Tables 4.1a and 4.1b identify discharge limits from October through May. The City's permit does not allow for any discharge to the Long Tom River from May 31 through October 31.

4.1.2 - Bacterial Limits

The permit requirements for bacterial limits differ from winter to summer. Table 4.1c shows the permitted discharge limits for bacteria. For winter discharge *E. coli* is used as the monitoring parameter and for summer land application, Total Coliform is monitored.

Table 4.1c

Bacteria Discharge Permit Limit		
Parameter	**Point of Discharge**	**Limitations**
E-Coli	River Discharge	Shall not exceed 126 organisms per 100 mL monthly geometric mean. No single sample shall exceed 406 organisms per 100 mL.
Total Coliform	Land Application	Shall not exceed 240 organisms per 100 mL in two consecutive samples and a 7-day median of 23 organisms per 100 mL.

4.1.3 - General Suitability Criteria

Two criteria used to measure general suitability of the effluent water are pH and temperature.

pH is the measure of the acidity of the wastewater. A value of 7 represents a neutral pH. Very low or high pH values can significantly impact the receiving stream and associated ecosystem. The permit requires that pH be between 6.0 and 9.0. Veneta has never had an issue or problem managing pH in the influent or effluent. There is no record of any deficiency related to the pH of the discharge.

Excessively high temperature in the discharge can also harm the stream ecosystem. Veneta has never had an issue with temperature being outside normal limits. The permit does not require the City to monitor temperature and no temperature data is available. DEQ helps to protect the summer stream temperature by not allowing discharge to the stream during the summer months.

4.1.4 - Other Permit Requirements

Additional regulatory criteria permit requirements in a large part relate to good housekeeping and reporting. The permit also prescribes the following:

- Specific discharge locations in the Long Tom River
- Prohibits discharge of untreated sewerage
- Prohibits the presence of chlorine in the discharge
- Requires balanced irrigation with crop requirements

- Prohibits discharge to the groundwater
- Outlines specific monitoring requirements on a daily, weekly, bi monthly or quarterly basis for:
  - Parameters shown above
  - U.V. Radiation Intensity
  - Irrigation depth
  - Nutrients

In addition to monitoring and reporting, the permit requires that the plant operations be certified and managed by an operator with a Level II license. Operator licensing requirements are prescribe in the Oregon regulations. (OAR 340-049).

4.1.5 - Biosolids Management

Biosolids are the solids recovered from the treatment operations. They have been historically called sludge but a trend in the industry that began in the 1970's has been to call these solids "biosolids" to reflect both their origin and suitability for reuse. The plant biosolids are held in facultative lagoons. The permit requires that the biosolids depth in each of the facultative lagoons be measured annually and that when the biosolids are scheduled for land application that a comprehensive chemical and biological analysis of their content is conducted (See Permit for testing list). When the biosolids are applied at a DEQ approved site; a record of the date, volume, locations of application shall be duly recorded. Anytime biosolids are removed from the facility a record of the land application process as well as any record of spills or equipment breakdowns are to be reported to DEQ.

4.1.6 - Annual Reports

In addition to the reports itemized above, the permit requires:

- The City prepare an annual program focused toward inflow and infiltration reductions and document the results of that program
- An annual summary of the reclaimed water system and
- Biosolids summary report when biosolids are land applied

4.2 – Future Regulations

Below we review future discharge standards and how the existing plant (or a plant of similar design) will perform in the future. Our understanding of the regulatory trend is also reviewed.

4.2.1 - Discharge Standards

It has been a consistent policy of DEQ that as a community grows its allowable discharge remains the same. That is, the permit limits shown in Tables 4.1a, 4.1b and 4.1c will be the City's discharge limit into the future.

4.2.2 – Plant Performance

The impact of this policy is illustrated in Table 4.2a. Based upon the allowable discharge of 130 pounds of TSS per day the existing plant must operate at a treatment efficiency of 94.4 percent to meet the discharge limit. A look at the influent characteristics of January 2007 we see that the plant's hydraulic flow nearly matched the design month and the plant needed to operate at a 92.1 percent efficiency to meet the discharge limit. (The plant's actual efficiency that day was 98.8 percent). Future treatment requirements are reviewed in detail in Chapter 5.

Table 4.2a

Comparison of Total Suspended Solids Treatment Efficiency Requirements					
Event *	**Flow (MGD)***	**Influent TSS Concentration (mg/L)**	**Influent TSS (lbs/day)**	**Allowable Discharge (lbs/day)**	**Plant Efficiency Required to Achieve Discharge Limit %**
Jan. 2007 Average Flow	0.936	277	1,639	130	92.1%
Design Month	0.92	303	2,324	130	94.4%
Future Average Day	1.236	340	3,504	130	96.3%
Future Peak Month	2.917	280	6,809	130	98.1%
Peak Day	5.46	250	11,380	260	97.8%
May 1-31	2.0	330	5,502	66	98.2%
* Selected Existing & Future Loads					

Our experience and literature documentation indicate with extended aeration system vendors our experience that the process guarantees for these plants is 15 mg/L for TSS. Although they can, and do, consistently produce an effluent less than 15 mg/L the suppliers limit their guarantee. If we examine the projected future flow rates illustrated in Table 4.2a for the peak month, peak day and May event scenarios we see the effluent quality requirements projected in Table 4.2b.

Well operated extended aeration plants such as Veneta's Biolac system are capable of, and do, consistently produce effluents of 10 mg/L or less. But, it is unrealistic to design for these plants to produce day in and day out, over a wide

variety of flow and solids loading conditions, a discharge less than 10 mg/L. As indicated in Table 4.2b an effluent consistently less than 6 mg/L will be required during the design year.

Table 4.2b

Effluent TSS Required (mg/L) to Meet Discharge Limit			
Event	**Flow**	**Allowable Discharge (lbs/day)**	**Required Plant TSS (mg/L)**
Peak Month	2.917	130	5.3
Peak Day	5.46	260	5.7
May	2	66	4

4.2.3 – Additional Parameters

When examining future facilities it is important to plan for these improvements in the context of a new and/or pending regulatory environment. The literature and discussion forums in the industry are filled with articles and speculation as to what regulations may be enacted to provide greater assurance of public health protection. In order to have a better understanding of DEQ's position on pending regulations we arranged a meeting with DEQ to review possible future regulatory environments. The questions reviewed in that meeting are included as Appendix E. The meeting was attended by two engineers representing Veneta and five staff members from DEQ. The DEQ representatives included the regional engineer a second engineer, an area manager, a permit writer and a permit enforcer. Key discussion points are reviewed below:

- DEQ does not anticipate any change in how collection system bypasses are managed. In the event that a five year storm even occurs it is preferable that the bypass point be a singe point. The agency recognizes that system overload points cannot always be managed. The meeting attendees did caution us they have received some criticism from EPA regarding management of overflows and that the existing bacteria rule is undergoing scrutiny and changes in it may impact how overflows are regulated.
- Future plant expansions, done entirely with City funds, may not require a comprehensive assessment and review of alternative technologies. However, if the City were to use State Revolving Loan funds (SRF) for any expansion then an agency review of alternatives would be required.
- Where, in the past, DEQ has required rigorous application of water reuse waters in accordance with sound agronomic principals the Department is realizing that overwatering for agriculture practices may be an acceptable practice. OAR 340 Division 55 rules apply.

- DEQ has recently been encouraging changes to the plumbing code that would allow for more aggressive water reuse including reuse water for in home non-potable applications.
- Although the literature and research have identified various esoteric, or trace, compounds as possible pollutants (i.e. endocrine disruptors, hormones, legal and illegal drug residuals, etc.) staff at DEQ do not see an immediate need or legislative push to monitor and regulate such parameters.
- In spite of recent Governor initiatives regarding the identification of and monitoring for "carbon footprint" and energy consumption trends, the staff at DEQ is not aware of any momentum to include publicly owned treatment plants in any monitoring or policing activity toward energy use.
- The meeting attendees did not feel that there was any regulatory momentum toward adding publicly owned treatment works to those entities requiring air pollution discharge permits. Such treatment works will still be required to comply with odor thresholds and similar criteria. However, OAR 340-215 does require wastewater treatment facilities with NPDES permits to register and report greenhouse gas emissions when such emissions exceed 2,500 metric tons per year. The City can go to www.epa.gov/cleanenergy/energy-resources/refs.html for more information.
- Their remains confusion regarding how DEQ staff and the regulations interpret the need for a facility plan. Veneta is currently operating under an approved facility plan developed in 1997. Routine expansions and upgrades can be initiated and approved under that facilities plan and a Biolac expansion may be considered as part of such a routine upgrade. However, if SRFs are used for any expansion then a facility plan update will be required.

4.3 –Water Reuse Regulations

Water reuse involves putting to beneficial use treated municipal wastewater. Common beneficial uses include agricultural irrigation; water for industrial cooling; commercial car washing; non-residential toilet and urinal flushing; and water supply for landscape impoundments. Oregon water reuse regulations specify different levels of treatment quality depending on the intended use.

Many communities in Oregon have begun to experience increasing, at times overwhelming, pressure on the quantity and quality of water available for drinking water. In a response to this, the DEQ amended its rules for recycled water in April 2008. These amended rules are found in OAR 340-055. These rules emanated from community leaders and concerned citizens who saw water supply issues as a threat to Oregon's "quality of life and the State's environmental and economic sustainability". The rule changes promote sustainable practices in Oregon by encouraging the use of recycled water for an expanded variety of beneficial purposes. The revised rules encourage innovation in treatment

technologies and removed some regulatory barriers to water reuse in Oregon. These rules open new doors of reuse opportunity that did not exist at the time the existing facilities were designed. Two very significant aspects include the allowance of use of treated effluents for wetland enhancement and reuse for non-potable water in occupied buildings.

The new rules help to protect and conserve the water resources of Oregon by recognizing the resource value of recycled water. Using recycled water for beneficial purposes improves receiving waters quality by recycling the quantity of effluent discharged from wastewater plants. By using recycled water in place of potable water, the peak potable water demand is reduced, conserving potable water supply.

The 2008 DEQ treatment criteria for water reuse revise DEQ's long held classification of effluent requirements based upon treatment levels (DEQ had defined treatment levels from Level I through Level IV). The revised treatment reuse classifications are now given different treatment class designations with the highest level of treatment and allowable water reuse designated as Class A. The implications of these revised water reuse standards are reviewed more fully, for Veneta, in Chapter 6.

Current Oregon regulations prohibit the use of recycled water for drinking water. However, using reclaimed water for aquifer recharge is not explicitly prohibited by Oregon's rules for recycled water use. OAR 340-55-0012 (7) (a) (F) defines artificial groundwater recharge (by surface infiltration or subsurface injection) as a "beneficial purpose" appropriate for the reuse of Class A effluent.

OAR 340-044-0015 (2) (f) prohibits injection of treated municipal wastewater directly into an underground source of drinking water. The following statement is taken from an April 7, 2008 Environmental Quality Commission memorandum and indicates that recharging an aquifer with highly treated sewage may be permitted if the treated effluent complied with drinking water standards.

> "***Regulatory barriers for aquifer storage and recovery and wetlands:*** *Through the rulemaking process, DEQ and the Water Reuse Task Force identified several regulatory barriers that restrict the use of recycled water for aquifer storage and recovery, and wetlands restoration and enhancement. DEQ's rules in OAR 340-044 for underground injection control (UIC) prohibit the injection of municipal wastewater directly into an underground source of drinking water. If injection were allowed, the water would have to comply with drinking water standards, and meet background water quality levels or a concentration limit variance. The need to revise the UIC rules has been identified to address overall UIC program implementation issues and recent legislation on fee authorization."*

We believe that the rules restricting such processes will be under review and will be adapted to reflect the changing posture of Oregon toward water reuse.

The regulations reviewed above provide the base for planning of treatment and reuse facilities in Chapters 5 and 6.

5. TREATMENT PROCESSES

Chapter Summary

This chapter reviews the existing treatment processes and identifies known or projected deficiencies in operation, sizing and/or performance. The chapter walks the reader through the wastewater treatment train and identifies current and future deficiencies that may occur over the 2010-2030 planning horizon. A graphic that illustrates the treatment plant process is provided in figure 5.1. The chapter is organized into the following sections:

5.1 Existing System Basis of Design

5.2 Future Plant Concept

5.3 Wastewater Treatment Plant Costs

As reviewed in chapter 2 the evaluation of the treatment processes is based upon a current (2007) population of 4,640. For 2030 analysis, the population is estimated at 9,660.

The existing Influent Flow Metering devices are adequate for the 2030 planning period if the recommended collection system alternative is implemented. An additional meter will be installed at the treatment plant to measure influent from the pump station network.

The influent pump station is currently 12% under capacity. Minor upgrades will be necessary as the expanding collection system will be mostly pumped and bypass much of the existing influent pump equipment.

The surge basin currently holds 4 million gallons (MG). In the planning criteria presented here, we recommend changes to pump capacity to eliminate the need to modify the surge basin. No mechanical aeration will be required, nor will the overflow weir require additional capacity.

The headworks screen requires replacement. Modifications of the hydraulic channel housing the screen are required. The rough screens below the influent pumps do not need upgrading. Flow splitting may or may not undergo changes.

The main treatment unit (the aeration system) has experienced loads above the design capacity. The treated plant effluent is well below, about 50%, of permitted BOD_5 and TSS limits. The entire plant is at around 85% of its population loading capacity. The Biolac system should function properly until 2014, however its capacity will need to double for the 2030 planning period. This will result in one or two new Biolac basins, depending on configuration. The aeration piping needs immediate repair.

The UV disinfection system is currently being operated beyond its original design expectation. The plant UV system cannot maintain appropriate disinfection. The disinfection system needs to be upgraded to conform to stricter effluent standards.

The effluent flume has adequate capacity. The planted areas designated for plant effluent uptake are a mix of grass and poplars. Current operations tend to overwater the grass. The poplar plantation has proven to be costly to maintain.

The sludge lagoon has sufficient capacity for the 2030 planning period. The sludge dredge owned by the city is not in use, and could be sold as excess property.

5.1 – Existing System Basis of Design

The wastewater treatment processes were designed to perform adequately under various operating conditions. The limits for these operational parameters are listed in Table 5.1a. The table shows the original design parameters for the treatment plant constructed in 2001.

Table 5.1a

Existing Plant Process Design Parameters		
Plant Process	**Design Operating Condition**	**Value**
Influent Flow Measurement	Million Gallons per Day (MGD)	0.07 – 10
Influent Pump Station	Million Gallons Per Day	1.25
Surge Basin	Million Gallons	4
Aeration Basin	Peak Flow (MGD)	1.25
Blowers	Standard Cubic Feet/ min	592
Clarifier	GPD/Square Foot	978 Peak
UV Disinfection	Summer - Total Coliforms Winter – *e-coli*	23 126
Water Reuse	GPM at PSI	300 at 88
Sludge Management	Facultative Sludge Lagoons	20 lbs VSS/1000 square feet/day

In sub-sections 5.1.1 through 5.1.5 the current operating conditions of the various treatment plant components are reviewed based upon the design criteria established for those treatment processes in the original plant design. Deficiencies are identified when anticipated future demand over the planning period exceeds the stated design capacity.

5.1.1 – Headworks

The headworks consist of five components that together manage the incoming waste stream:

- Influent Flow measurement
- Screw Pumps
- Surge Basin

- Screening
- Flow Splitting Structure

The requirements for any future headworks modifications are strongly dependent upon the selected collection system alternative. For the discussions provided below we have assumed that Collection System Alternative 1, from Chapter 3, will be implemented.

5.1.1.1 – Influent Flow Measurement

Influent flow is measured in a specially designed manhole that includes a 12-inch Parshall flume. The flume is located on the driveway approach to the treatment plant gate, about half way up the hill toward Sertic Road. It has the capacity to measure flows from 0.07 to 10 MGD. A small flow metering station is also scheduled to measure flow from approximately 26 homes in the Shadow Ridge Subdivision, and contributory areas to West Hunter. These homes tie in to the collection system downstream of the main flow meter. This auxiliary flow unit may be required to meter 91 homes under future conditions. The flow from this station needs to be added to the flow from the existing flume in the daily monitoring reports.

Peak hourly flows over the planning period are estimated to be below 6.6 MGD. Thus, under Collection System Alternative 1, the influent flow measurement facilities are satisfactory. Alternative 1 specifies new pump stations that would be equipped with flow metering devices. The pump station flow does not travel through the existing meter. We recommend installing a new influent flow meter at the treatment facility to measure flow coming from the pump stations.

5.1.1.2 – Influent Pump Station

The influent pump station (IPS) lifts wastewater coming into the plant from the lowest point of the gravity sewer system up to the treatment process. The wastewater must be lifted to an elevation high enough so that it can flow through the treatment plant without additional pumping.

The IPS consists of two screw pumps each with a rated capacity of 1.25 MGD. One of the pumps is configured to operate at the rated capacity. The second operates at a speed to deliver a flow rate of about 0.95 MGD. The operator prefers the slower speed and output for the second pump, as it provides steadier input during lower flow periods. This increases operator flexibility and may reduce power consumption. Veneta does have on hand the means to change the pump speed and provide 1.25 MGD with the second pump with a minimal effort.

Current maximum monthly flows are estimated at 1.4 MGD. Thus, the existing screw pumps are about 12 percent under the required capacity to service maximum month conditions using only one pump. The operator has used surge basin capacity and manual operation of the second pump to manage peak flow conditions experienced during 2006 and 2007 storm events. Upgrades to the influent pumps should be implemented soon. It may be possible to change pump speed by changing the gear ratios and achieve a higher pumping capacity. The City should work with their

engineer and equipment vendor to determine speed and pumping capacity to limits based upon the screw design and motor output.

5.1.1.3 – Surge Basin Current Conditions

A surge basin is used to "equalize" or level flow rate variations into the plant by storing the extreme influent flows experienced during storm events that exceed the treatment plant capacity. When headworks capacity is exceeded, flow is temporarily stored in the basin and returned to the plant as the flow recedes. The flow equalization reduced capital costs of the plant because the components are sized for lower flow rates than the peak flow. Veneta's surge basin has not experienced overloading or deficiencies since the operation began in 2001.

The surge basin was originally sized to store four million gallons based upon the results of a hydraulic analysis of flow records. A safety factor of approximately 20% was applied to the original design. Flow records during a peak month in 1996 were used to estimate the peak instantaneous, peak week, and peak monthly flow occurring in a single month. The design intent was to balance regulatory requirements, site conditions, and treatment plant capacity, with equipment capital costs, construction costs, operating flexibility and the need to protect the treatment plant from excessive hydraulic loading.

Proper sizing of a surge basin requires analysis of the period of time required for basin recovery. For the City of Veneta, basin recovery has been the confining factor. When a series of significant storms arrives and the influent flow rises, the surge basin takes the influent that is in excess of the screw pump capacity. Past experience has shown it can take over a week to empty the surge basin following large storms.

Capacity Analysis

An analysis of surge basin capacity was conducted to identify potential deficiencies. The worst period of storm input known for the wastewater treatment plant was from mid-December 2005 through mid-January 2006. No flow data was available for this period, so the surge basin capacity analysis used a theoretical scenario which requires the basin to handle a 10 percent excess flow from the collection system for five days followed by the peak day which includes the peak hour. Following this storm event, it was assumed that another 20 days occurred at the maximum monthly flow. Table 5.1b shows the surge basin capacity required to accommodate these hypothetical conditions.

Table 5.1b

Surge Basin Capacity Requirements 2007 (Assumes Max Monthly Flow @ 1.4 MGD w/ Pump Capacity of 1.25 MGD)		
Factor	**Description**	**Million Gallons Required**
Flow 1	5 days @ 110%	1.49
Flow 2	Peak Day	1.38
Flow 3	Peak Hour	0.13
Pump Shortage	20 Days	3
Surge Basin Need		6
15% Safety Factor	0.9 MG	6.9

Standard engineering practice is to apply a 10-20 percent safety factor to the recommended design size. We have added a 15 percent safety factor to establish a recommended surge basin need of 6.9 million gallons. The calculations for the basic surge basin size assume nearly ideal operating conditions and do not account for decrease in available surge basin volume caused by ongoing precipitation.

Based on this assessment, the surge basin may experience a capacity deficiency during the planning period of almost 3 million gallons.

Surge basins often include provisions for mechanical aeration to prevent odors. The experience at Veneta has been that the surge basin is sufficiently large, the detention time is typically short, and the waste is of low organic strength and does not lead to odor generation. No records of dissolved oxygen levels in the surge basin are available and therefore we cannot predict if odor issues are imminent. Experience suggests that upgrades to the surge basin with aeration equipment are not necessary.

Surge basin capacity can be expanded by adding an additional cell or cells to the east or north that are hydraulically connected to the existing basin. The additional cell required for 3 million gallons of additional storage would require approximately 2.3 acres. It is possible to add cells in a modular approach, added as growth and future flows dictate. See Section 5.9 for a review of surge basin requirements for future facilities.

A component of the surge basin under review in this report is the overflow weir. It is the design element that controls the water level at which excess sewer flow is diverted to the surge basin. The current capacity of this weir is 7.1 MG. Therefore the overflow weir has adequate capacity for current and future needs.

5.1.1.4 – Screening and Grit Removal

The screening system removes objects from the waste stream such as rags, large table scraps, paper, plastics, metals, wood, grit, all which can cause inefficiency and damage to the down stream treatment process. Screening is a vital component of the treatment plant. A condition of the process guarantee offered by the aeration system supplier is that the plant has a screened influent. Veneta's screening system consists

of parallel channels. The South Channel houses a rotating drum screen and the North Channel is a bypass channel. A rough manually cleaned bar screen (second screen) is installed at the plant in front of the screw pumps. This rough screen has two inch openings installed to protect the screws from large objects that might jam or damage the screw flights.

The primary screening is performed in the south channel by the rotating drum screen manufactured by *Andritz* that has a rated capacity of 2.5 MGD. The bypass channel has a three quarter inch spacing bar rack for rough screening. This channel provides a route for the waste stream when the primary screen requires maintenance. Normally this channel is closed off because operation of this channel leads to quick buildup of screenings and requires vigilance and significant labor to keep an even flow. When not obscured the bypass channel has a capacity of over 10 MGD.

The Andritz screen was installed in 2001. It has not performed reliably for Veneta. Numerous operation and maintenance issues have been caused by failing parts which have resulted in costly repairs and disrupted operations. The first elements to fail have been solenoid valves which are electrically activated. Opening of the solenoid valves provides for washing of the rotating drum screen with recycled, treated wastewater. In 2008 the downstream mounting flange of the screen wore completely through and required over $10,000 for repairs. The *Andritz* screen has remaining capacity and with the 2008 repairs, some remaining service life. However, given the operational history, it is a high priority of the operators to find an alternative screening system. Replacing the screening system is recommended as a good screen protects the plant and prevents disruptions.

Several alternatives are available to provide screening for the current and future treatment processes. The channels that house the current screen were designed specifically for the Andritz screen. A different system may require significant modifications to the existing channel or a completely new hydraulic structure.

The rough screens at the front of the plant that protect the screw pumps are not a hydraulic deficiency to the plant. They do require operator attention for cleaning of rags and debris, but they do not require upgrading or modification.

5.1.1.5 – Flow Splitting

Following the drum screen, the headworks have a flow splitter structure that allows the operator to channel incoming flow to either or both aeration basins. Each channel has a rated capacity of 1.25 MGD. These have adequate capacity for the existing flow conditions.

The flow splitting channel may or may not undergo modifications under the future development plan. This depends on the layout and nature of future plant upgrades and modifications.

5.1.2 – Aeration System

The aeration system consists of the blowers, aerators, pipe network and diffusers which provide oxygen for aerobic treatment in open basins. Veneta's wastewater plant uses Parkson's proprietary system called the *Biolac* process. The *Biolac* elements include the air (oxygen) delivery system, separation wall, and clarifier and overflow weir. The plant relies on extended aeration, modestly concentrated mixed liquors and long sludge age to produce a quality effluent.

The air is supplied via electrically powered, mechanically driven lobed blowers. The plant has three blowers at 20 horsepower each and a rated capacity of 697 standard cubic feet per minute (SCFM) against a discharge pressure of 5.4 PSIG per blower. The aeration units, in conjunction with the diffusers provide two functions: mixing and oxygen. Mixing is maintained in the basins with one blower operational. The oxygen required for treatment is a function of organic load.

The air from the blowers is delivered to the aeration basins through 10, 8 and 4 inch diameter ductile iron pipe. The air, after being compressed by the blowers, is often delivered to the basins at high temperatures. The air temperature in the pipe can exceed 200 degrees F. The extreme high temperature, the existence of ozone and sometimes petroleum particles in the air lines can all contribute to deterioration of SBR gaskets in the air delivery lines. This has occurred in Veneta. All of the pipe joints in the air delivery line have experienced deterioration and this has contributed to significant loss of efficiencies in the air delivery system. It is estimated that as much as 50% of the air destined for treatment never makes it to the treatment basins because of gasket failure. Weber Elliott contacted various suppliers of in-situ pipe repairs and we were not successful in finding a supplier that felt their product would handle the high temperatures. Nonetheless full repair of the aeration piping is an immediate concern and should be budgeted and implemented immediately.

The oxygen delivery system for the *Biolac* system was designed for a daily average BOD_5 load of 1097 pounds to produce an effluent BOD_5 of 10 mg/L. The system is rated to treat peak instantaneous flows of 2.6 times the average flow for two hours per 24-hr period. A maximum daily BOD_5 load to the plant is 1243 lbs per day.

5.1.2.1 – Input Loads

Table 5.1c shows average monthly input loads to the plant for year 2007 and two selected high load days (January 29, 2008 and April 8, 2008). The table shows that the influent organic loading averaged 1214 pounds per day (97 percent of the design load). The table also illustrates that, at times, the plant influent load exceeds the average design value. Exceeding the maximum daily loading of, 1243 pounds per day of BOD_5 occurred during about half of the months in 2007.

Table 5.1c

Influent Load					
Month	**FLOW (Gal)**	**BOD_5**	**TSS**	**Lbs/day BOD_5**	**Lbs/day TSS**
Jan, 2007	0.936	164	210	1280	1639
Feb, 2007	0.815	227	296	1542	2011
March, 2007	0.729	192	338	1167	2054
April, 2007	0.493	290	370	1192	1521
May, 2007	0.466	458	615	1779	2389
June, 2007	0.362	407	485	1258	1498
July, 2007	0.343	271	295	775	844
Aug., 2007	0.312	271	249	705	648
Sept., 2007	0.324	521	658	1407	1777
Oct., 2007	0.431	422	538	1516	1933
Nov., 2007	0.527	218	271	958	1191
Dec., 2007	0.641	187	252	999	1347
Average 2007				**1214**	**1571**
Jan. 29, 2008	1.248	178	308	1852	3205
Apr. 8, 2008	0.448	1480	1080	5528	4034

5.1.2.2 – Plant Performance

The plant performance is a more critical evaluation factor than the design influent loading. Tables 4.1a, and 4.1b, show that monthly average BOD_5 and TSS discharge permit limits are 130 pounds per day for winter operations and 44 pounds per day in May and October with peak allowable BOD_5 and TSS concentrations limited to 15 mg/L during these transition months. Table 5.1d shows effluent values for BOD_5 and TSS for 2006 through 2008 winter operations and Table 5.1e shows total effluent loading for transition months (May and October) 2006 and 2007.

The plant has experienced BOD_5 and TSS loads exceeding the design values. The worst month recorded was February 2008, where BOD_5 and TSS output averaged 55 and 63 pounds per day respectively. Although the plant service population is at 85 percent of the design capacity and the design loads have been exceeded, many times, the actual plant discharge is less than half of the permitted 130 pounds per day. For the transition months of May and October where the monthly discharge limit is 44 pounds per day, the plant is producing an effluent that is about half of the permitted allowance (i.e. 22 pounds/day).

Table 5.1d

Effluent Discharge Winter Month Average			
Year	**Month**	**BOD_5 (LBS/Day)**	**TSS (LBS/Day)**
2006	Nov.	0	19.6
	Dec.	10.8	15.6
2007	Jan.	17.6	19.6
	Feb.	19	32.7
	March	27.9	35.1
	April	9	13.8
	Nov.	18.6	54.3
	Dec.	7.3	22.5
2008	Jan.	25	26
	Feb.	55	63

Table 5.1e

Effluent Transition Month Average			
Year	**Month**	**BOD_5 (LBS/Day)**	**TSS (LBS/Day)**
2006	Oct.	5.2	14.5
2007	May	23.9	19.1
	Oct.	0	15.7
2008	May	14.5	22.2
Note: Where “0” is shown BOD_5 levels were non-detect.			

In summary, the population load to the existing plant is at 85% of capacity. However, the plant’s performance is such that the effluent quality is well below, about 50%, of the permitted discharge limits. Overall plant performance will fall off with increased hydraulic load (i.e. expanded growth). We predict that the plant will continue to provide an effluent that meets and or exceeds permit requirements up to the design population level (5471). Based on past performance, it is reasonable to expect that the plant may perform adequately for a population approaching 6000.

An analysis of past performance applied to the 2010-2030 planning period predicts that the existing Biolac system will perform adequately until around the year 2014. To provide a system that serves the entire period, the system would require a doubling of current capacity, scheduled near 2014.

It is assumed that the new treatment system would roughly mirror the old system in terms of basin size and infrastructure requirements. A design for increasing the

capacity of the *Biolac* basins could employ either one or two additional basins. In Section 5.9 we review the cost for a two basin expansion.

5.1.3 – Clarifiers

The clarifier design comes as a part of the *Biolac* treatment process. Therefore its performance is integral to the discussion reviewed in Section 5.3.2. That is, the plant output is from the clarifiers and a result of the combined performance of the aeration system and clarifiers. The clarifier design rate is established at 1.38 MGD for the peak hour or 978 gallons per day per square foot (gpd/sq ft). The clarifiers are adequately sized to meet current loading conditions and are expected to perform along with the aeration basins until 2014.

Based on the hydraulic loading criteria reviewed above, and a history of the operator using both screw pumps to assist in storm recovery, we believe that the clarifiers have experienced short term loads in excess of the 978 gpd/sq ft. The performance of the clarifiers is hampered in part by the tendency of the clarifier weir to bend during high flows. This bending leads to accelerated up flow rates within the clarifier. This condition is further exacerbated when dual screw pumps drive so much water through the plant that water pours into the overflow cylinders within the clarifiers. The unbalanced load across the weir contributes to the poorer TSS removal rates shown in the performance data. This is substantiated when one compares effluent BOD_5 to TSS ratios. Average effluent BOD_5 to TSS ratios calculated from January 2005 through May of 2008 showed TSS at 1.67 time's effluent BOD_5. Average BOD_5 to TSS ratios during winter months (December through March) were 1.83. In other words, the plant tends to discharge 10 percent more TSS, relative to BOD_5, during winter months.

This may indicate that the clarifier over topping contributes to the performance deficiencies beginning to show up and discussed in Section 5.3. With the Biolac system the clarifier is structurally and systematically integrated with the aeration basins. Each individual aeration basin has a clarifier. For planning purposes, any upgrade or new construction of a clarifier would be accompanied by corresponding aeration basin construction.

5.1.4 – Disinfection

The Ultraviolet (U.V.) disinfection system operates in a cast-in-place concrete channel 26 feet long by 21-inch wide by 42-inch deep that contains the U.V. lamps. The amount of U.V. light (dosage) that can be received by micro-organisms is dependent on light intensity, and clarity (transmittance) of the wastewater entering the system. The main factor affecting transmittance is total suspended solids (TSS). Light intensity diminishes as the lamp ages.

The level of disinfection required is set by DEQ in the NPDES Waste Discharge Permit. The current system is designed to disinfect effluent to a "Level II" standard, which is typical for secondary wastewater treatment plants. The standard allows for river discharge in the wet weather months and establishes more strict conditions for irrigation in dry weather months. The regulations have since been updated and the classifications have changed. The current U.V. system treatment capacity most

closely corresponds to the new "Class C" standard. (See Chapter 4 for a review of regulatory requirements.) The discharge is not allowed to exceed the limits indicated in Table 5.1f.

Table 5.1f

UV Design Parameters		
	Winter	**Summer**
Peak Process Flow	1.25 MGD	0.53
Retention Time	6.96 Seconds	16.4 Seconds
Dose Required	30,000 mw. SEC/cm2	Not determined
Suspended Solids	11 mg/L	11 mg/L
UV Transmittance	60%	60%
Disinfection Organism	*E. Coli*	Total Coliform
Disinfection Standard	126 Geometric Mean/100mL	23 -7 day median/100mL
Maximum Sample	406 MPN/100mL	240 MPN in 2 consecutive samples

The equipment installed for service at the plant was designed to provide only disinfection to the Wet Weather Standard (i.e. 126 MPN per 100 mL. In 2001, Veneta received a letter from Trojan Technologies Inc. (the U.V. supplier) stating that they would not guarantee that the system would achieve the total coliform standard (23 total coliform forming units per 100 mL) unless the flow was less than the stated design flow and the turbidity was less than 2 NTU's. The City recognized the pending inadequacy of the disinfection system by authorizing the necessary studies and design of a modified system in autumn of 2007. That design authorization, however, has not proceeded pending the outcome and recommendations of this master plan. There was a short period in 2008 where the U.V. system failed to meet the permitted disinfection standard.

As reviewed above, influent to Veneta's treatment plant occasionally exceeds the design capacity of the plant and, by extension, the U.V. disinfection system. The U.V. disinfection system has historically performed very well within the design limitations, but the U.V. system lacks sufficient capacity to disinfect the wastewater effluent to the required standard for the design life of the plant without operating both banks of lights. The second bank of lights was designed for system "redundancy" in case of failure of one bank and not as standard operating mode.

The current disinfection system is not capable of meeting the summer conditions noted in Table 5.1f.

Planning for upgrades to the disinfection system will be heavily dependent on the course the City elects to take with respect to the disinfection classification standard.

If the City pursues “Class A” effluent, which offers the most opportunities for beneficial uses, the design parameters would be considerably different than for the current “Class C” standard. This issue is more fully discussed in Chapter 6.

5.1.5 – Effluent Management

Effluent management facilities include the water recycle system, effluent flow measurement flume, and river discharge and irrigation storage pond land application reuse area.

5.1.5.1 – Water Recycle System

Following disinfection treated effluent collects into an approximately 5 foot by 5 foot by 3 foot deep concrete pool (approximately 550 gallons). From the bottom of this sump two pumps pressurize the treated water to about 80 psi. It is stored in a large steel pressure tank and distributed around the treatment facilities for wash down water, landscape maintenance and odor control. Its most significant use is spray nozzles on the influent screen. These high pressure pumps have not had a long term service record and the operators are concerned about bacteria re-growth in the large sump. However, these pumps are about the only suitable means of providing adequate pressure so no changes are proposed. The holding tank can be reconfigured to use air pressure instead of a bladder tank which may relieve some operational challenges.

5.1.5.2 – Effluent Flume

The effluent flume measures the total plant discharge. Measurement occurs after the recycle water is used within the plant and after disinfection. The existing flume configuration is limited to a measurement of about 1.9 MGD before the flume backwaters begin to interfere with the overflow of the U.V. system. Minor changes in configuration that will occur with the U.V. system changes will allow the existing flume to measure up to 2.5 MGD. The surge basins, screw pump limitations, and the aeration basins all modulate peak flows. Therefore a 2.5 MGD flume capacity is adequate at this time.

5.1.5.3 – River Discharge

The river discharge is via an eighteen inch diameter ductile iron outlet pipe. The pipe is configured with a flanged outlet to allow for addition of a diffuser, if required. The calculated capacity of this outfall is 1.25 MGD when the river is at its 5 year flood elevation.

5.1.5.4 – Irrigation Storage

Following disinfection the plant has a 2.3 million gallon storage pond that holds treated water until the irrigation pump move the water to the land application reuse area. The size of the pond was originally configured based on minimizing earthwork and assuring that the pond had sufficient capacity to hold water during October when land application plant uptake was minimal and river discharge was not allowed. Now that the permit allows for controlled discharge during October the 2.3 million gallons is adequate for future short term storage of treated effluents.

The pond is lined with a polyethylene (PE) liner. On occasion subsurface conditions will be such that in selected portions of the pond the liner is subject to floating. These floating conditions seldom are greater than a couple of hundred square feet. They do reduce the capacity of the reservoir but have not been an operational or process inhibitor. Slow filling in the spring accompanied by working the bubbles to the edge and allowing for the gas venting system to eliminate trapped gas would prevent the development of significant bubbles in the future. But as this requires significant labor and the liner is not subject to increased risk of failure such effort is not required.

5.1.5.5 – Land Application Reuse Area

Veneta currently has 100.5 acres of land north of Highway 126 that is used as the effluent receiving area from June through September and portions of May and October depending on Long Tom River conditions and effluent quality (See Chapter 4). The city requested an analysis of the effluent application area requirements in 2005. Midway through that analysis DEQ negotiated with Veneta regarding effluent discharge requirements for the critical months of May and September. The negotiations resulted in revised permit conditions. The revised conditions are evaluated below.

Veneta's current holdings managed for effluent application include:

- 18.7 acres planted in Hybrid Poplars 6 to 8 years old
- 20.2 acres of designated buffers and access routes
- 61.6 acres of grass

The capacity of the effluent application area is evaluated based on estimates of 70 GPCD resulting in current average summer flow of 0.33 MGD. The effluent application area manages this flow from June 1 through September 30.

The effluent application area may also receive treated effluent during the months of May and October. During dry May's and October's, there will be available capacity both from soil moisture capacity and crop uptake. This will allow the operator to balance land application rates with river discharge to maintain river discharge limits within the permit requirements.

The ability of the effluent application area to accept treated effluent is dependent on the ability of the receiving crop to utilize the applied water and the amount of water contributed from other sources. For Veneta, the other sources are rainfall. Oregon's land application regulations, prior to 2008, requires that the amount of water applied for reuse shall not exceed the plant's ability to use the water. This regulation helped to assure that treated effluents do not penetrate below the root zone and impact groundwater.

The facility plan prescribed effluent management plan provided for Veneta to plant a rotation of poplar trees over the 100 plus acres north of Highway 126 that would have the entire area planted by 2018. Management of a poplar plantation quickly brought three issues to the forefront.

- Weed management was a costly and labor intensive activity
- Without an aggressive weed management program the solid set aluminum sprinkler network experienced significant corrosion and
- Planting success was very poor in wetland areas

Because of these issues, the City has not planted any poplars since 2003; the ground sprinkler system was replaced, and application to grass and hay has been used for water uptake. Table 5.1g shows that the existing blend of poplars and grass has an uptake capacity of 85.1 acre feet. The treatment plant generates 122.1 acre feet of water in June through September. Thus, in a summer that represents the design conditions, the City over waters by as much as 37 acre feet (just over 4 inches). (Note that none of the summers since the plant construction have represented the wet conditions of a design summer). If the entire planting area of 80.3 acres were planted and managed as poplars then there would remain 23.2 acre feet of capacity (See Table 5.1h). This means that the poplar area would be at about 85% of capacity which is just in line with the ratio of plant population loading and the original design value of service for a population of 5471.

Table 5.1g

Assessment of Current Land Requirements					
	Uptake Rate (ft)	**Rainfall (inches)**	**Net Uptake (ft)**	**Number of Acres**	**Acre Feet of Uptake**
Poplars	2.46	7.82	1.81	18.7	33.9
Grass	1.48	7.82	0.83	61.6	51.2
				Sum	85.1
				Required	122.1
				Deficiency	37.1

Table 5.1h

Land Requirements if all Poplars					
Poplars	2.46	7.82	1.81	80.3	145.3
				Required	122.1
				Reserve	+23.2

The existing land application reuse area does not have sufficient wastewater management capacity as currently operated. Because the solution to this deficiency is related to the selected water reuse alternative this matter is further discussed in Chapter 6.

During a tour of the Wilsonville poplar plantation farm in 2006 the City learned that the planting pattern used in Veneta is nearly optimal. The Veneta planting pattern

was 7.5 feet between trees and 15 feet between rows. The row spacing allows for mechanical equipment to get between the trees for weed management and the tree spacing seems to be just right in that closer tree spacing tends to result in top damage during wind storms (tree tops tend to hit each other) and wider spacing may reduce water uptake.

The 2001 poplar planting was originally scheduled for harvest in year 2012 in order to avoid the cost and regulatory burden of compliance with the Oregon Forest Management Act. The Forest Management Act has subsequently been amended (2008) and there are some specific exclusions to a 12 year harvest mandate for certain siliviculture management plots that allow up to a 20 year harvest cycle. As this is a particularly unique area of management and land use law Weber Elliott contacted a forest consultant – Camas Creek Resources, LLC. to provide guidance on future management of the poplar plantation. We recommend the City consult with such a firm to complete a study that evaluates the following:

- Conduct a physical inventory of the stand to assess production and health.
- Conduct a silvicultural assessment and prepare recommendations for treatment to maintain the biological function of the plantation. Provide recommendation on schedule of treatments.
- Prepare a harvesting plan with recommended schedule of operations and cost and revenue estimates.
- Identify the requirements for a regeneration program.
- Review and assess the stand and harvest program as it relates to the Oregon Forest Practices Act
- Provide a professional opinion as to how federal and state wetland regulations may impact implementation of the plan.

It is recommended that the City seek the advice of someone experienced in the nuances of the Forest Management Act regarding appropriate protocol for harvest. It is known that once the hybrid poplar trees reach a certain age and height that they are much more susceptible to wind damage and disease. A qualified forestry consultant would guide the City toward the proper time to harvest. Preliminary estimates of costs to prepare such a lot assessment and harvest plan are $7,000.

A review of a 2002 regional pricing study indicates that chip prices have ranged from $72 to $97.50 per dry ton. Based upon estimates of 36 dry tons per acre and 85% yield it is estimated that the poplars trees may be worth a minimum of $2200 per acre at harvest time. Of course this gross revenue will be adjusted by market prices at the time of harvest and the cost of harvesting. It is estimated that replanting costs will be $6,200 per acre. The net planting costs are estimated at $4,000 per acre and two plantings of 9 acres cash are included in the program. Thus, $36,000 for years 2012 and 2014 is included in the capital improvement plan (Chapter 7) for these years.

5.1.6 – Sludge Management

The existing sludge management facilities include facultative sludge lagoons and a sludge dredge.

5.1.6.1 – Facultative Sludge Lagoons

The City has two facultative sludge lagoons. They are sized identically with an operating capacity of 0.46 million gallons, a 5.5 feet normal operating depth and a maximum depth of 6.5 feet. The lagoons are designed for a surface loading of 20 pounds of volatile solids per 1000 square feet per day. The 17,000 square foot lagoons are currently loaded at a rate of 5.4 pounds per 1000 square feet per day and are predicted to be loaded at 12.3 pounds per 1000 square feet per day when the plant serves a design population of 5471. There is no current or anticipated deficiency with the facultative sludge lagoons.

5.1.6.2 – Sludge Dredge

The City purchased a sludge dredge and operating control system as part of the 2001 upgrade. The City has found that it is more convenient to hire a firm experienced in sludge management for sludge removal and therefore does not use the sludge dredge. If the City is going to continue to contract out the sludge removal and hauling function then this dredge could be sold as excess property.

5.2 – Future Plant Concept

It is beyond the scope of work for this project to develop a mix of alternatives for evaluation of plant treatment work and processes to serve the 2030 wastewater treatment requirements. In this section we review the design criteria established for each of the plant processes and outline basic construction and upgrade requirements for development of a capital improvement plan. Figure 5.2 shows, at a concept level, the various elements that would be included in a future plant layout. Estimates for these improvements are provided as the concepts are developed. The estimates are summarized and incorporated into the Capital Improvement Plan in Chapter 7. Appendix C provides more detailed estimates of costs. Note that the costs shown in Table 5.2a and Table 5.2b include an allowance for engineering and contingencies.

Given the historical and excellent performance of the Biolac process, this document recommends that the City continue to use the Biolac system to accommodate system expansions required as part of this plan and further upgrades.

Any programmed expansion does require DEQ plan review and approval. Based upon discussions with DEQ, a programmed expansion using the Biolac process will likely be considered congruent with the existing approved facilities plan. However, should the City pursue any State Revolving Loan Funds (SRF) or federal funding support for plant expansions a Facility Plan may be required. It is recommended that prior to developing the scope and pursuing a project that DEQ be asked to review the project content to ensure that all appropriate planning documentation is in order.

5.2.1 – Headworks

The plant requirements for service to 2030 are prescribed in the regulations and outlined in Chapter 4. The plant upgrades will include modifications to the influent flow measurement system, influent pumping system, screening and flow splitting.

5.2.1.1 – Influent Flow Measurement

The Parshall flume located north of Sertic Road will provide adequate service for the main Sertic trunk line. For the future pressure line (collection system alternative), it is possible to measure all of the additional flow contributions by the installation of discharge flow meters at each pump station. However, it is recommended that a distinct measuring device be installed as part of the pressure line system outlined in collection system Alternative 1. The selected technology would likely be a submersible transducer with velocity sensors and a SCADA relay to the plant. The best location for the added influent flow meter would be at newly configured headworks associated with bringing the pressure line into the plant. Appropriate software would integrate the two different flow signals and provide a total for input into the DMR.

5.2.1.2 – Influent Screw Pumps

Based upon the proposed implementation of Alternative 1 for the Collection system planning we know that much of the future wastewater input to the treatment plant will come to the plant through the wastewater pressure line. For a peak day flow of 5.4 MGD the future operation projection indicates that 3.3 MGD would be delivered by the collection system pumps and the remaining 2.1 MGD would be delivered through the screw pumps. This represents 61% of the peak flow.

A cost effective pump upgrade at the influent screw pumps would involve a balance of pump capacity and surge basin size such that the existing surge basin could continue to function and the pumps would be sized to accommodate peak flow. Table 5.2a shows the pump station capacity sizing required to preserve the capital invested in the existing surge basin.

Table 5.2a

2030 Pump Capacity (Max Monthly Wet Weather Flow @ 1.77 MGD w/ Pump Capacity of 1.7 MGD		
Factor	**Description**	**Million Gallons Required**
Flow 1	5 days @ 110%	1.24
Flow 2	Peak Day	0.33
Flow 3	Peak Hour	0.14
Pump Shortage	Remainder of Month	1.4
Basic Surge Basin Need		3.10
15% Safety Factor	0.46 MG	3.56

From Table 5.2a the screw pumps must have firm capacity of 1.77 MGD. In order to provide redundancy this would require a third screw pump with a capacity rating of 0.52 MGE (1.77 MGD minus 1.25 MGD = 0.52). As an alternative, a conventional centrifugal pump could be added that meets these capacity requirements. That decision can be made during the preliminary design phase. For cost estimating we assumed that a parallel screw pump would be added to the existing pump facility. Preliminary engineering may determine that another pump arrangement is better for the City.

The Capital Improvement projects associated with the influent screw pump station include the following:

- Addition of a third pump with minimum capacity of 0.52 MGD
- Provide sun screen or shelter over pumps to minimize expansion and damage to flights.

5.2.1.3 – Surge Basin

Surge basin sizing is reviewed in section 5.9.3 in the context of pump sizing. With appropriately sized pumps the existing 4.0 MG surge basin will provide adequate service and no changes are included in the CIP.

5.2.1.4 – Screening

The City received a manufacturer's proposal on possible screening systems in early 2008. As outlined in section 5.2.4 it is the desire of the City to use different technology for future screening. The influent acceptance channel will need to receive a comprehensive redesign to accommodate the existing screw pump input and future input from the pressure lines and screw pump changes. There are two alternatives for this upgrade. One would provide for a single screen placed into a revised headworks channel or the second alternative would provide for two screens – one to handle screenings from the screw pumps and a second to handle screenings from the pressure sewer line. Table 5.2b provides an estimate of cost for headworks changes for the dual screening system. During a pre-design phase of the headworks it may be determined that a single screen is satisfactory and some savings may be possible.

5.2.1.5 – Flow Splitting

Channel and approach changes will be required to accommodate revised influent pumping and screening. Reconstruction of the influent flow splitting facilities will be done to provide for allocation of proportions of wastewater into future basins. The flow splitting system will be designed with a capacity of 5.4 MGD and configured to accommodate flow coming from the pump stations in the collection system and from the screw pumps.

The costs associated with modifications to flow splitting are minor when initiated as part of the overall program changes required for pump station modifications and screening.

Headworks Summary

All of the headworks facilities need to be planned and integrated into a single unifying design document. The design document needs to define the energy requirements for the entire plant under the new flow criteria and ensure that flow and energy conditions are fully satisfied prior to proceeding with headworks modifications and design of the system expansion. We have allocated $58,000 in the C.I. P. for this study that will establish the hydraulic performance criteria for the plant upgrade.

The cost summary for the headworks elements of the facility upgrade are shown in table 5.2b.

Table 5.2b

Summary of Headworks Costs	
Preliminary Engineering Assessment	$58,000
Flow Measure	$68,200
Pump Modifications	$242,400
Surge Basin Change	$0
Screening	$299,100
Flow Splitting Structure	$7,300
Total	$675,000

5.2.2 – Aeration and Clarifiers

Of immediate concern from a capital budget standpoint is the repair of the air lines for the treatment plant. We estimate $12,000 is required for repairs and should be budgeted for immediate implementation.

Parkson Corporation provided a cost estimate for material and equipment to mirror the existing aeration and clarifiers. Based upon that estimate and allowing for placement of the new twin aeration basins north of the existing basins we have estimated an installed cost of $2.04 million for upgrades to the *Biolac* system.

5.2.3 – Facultative Sludge Lagoon

Placement of aeration and clarifier additions at the location of the existing facultative Sludge Lagoon will require relocation of the sludge lagoons. It is the assumption of this report that relocation of the sludge lagoons is more expedient than installation of expanded treatment capacity at another location given all of the conveyance piping and controls required. Sufficient area for added facultative sludge lagoon is available east of the existing facilities. Based upon this land availability we have estimated the cost of constructing new facultative sludge lagoons at $736,000.

5.2.4 - U.V. Disinfection

As stated in section 5.5 the existing U.V. disinfection system is at capacity. Immediate upgrades are required. Those upgrades need to accommodate the future disinfection system requirements. Additional disinfection applications and

technologies are required for an expanded wastewater reuse program. The additional requirements to disinfect to a level that support total water reuse are reviewed in Chapter 6.

An estimate for meeting the immediate needs for disinfection technology material and equipment is $391,100. The upgrades include the materials, equipment, supplies and structures to house all of the U.V. channels, and light banks as well as power and control upgrades. In addition, an estimated $270,000 in upgrades would be required at the time the Biolac system expansion occurs. The new system would be designed basically as a three bay system so that any one channel could be out of service to ensure redundancy.

5.2.5 – Reuse Upgrades

The existing sump and pump should adequately serve on site reuse expansion needs. We have allocated $5,000 for pressure tank changes and other service upgrades, and included these costs as part of the plant expansion.

5.2.6 - Irrigation Pump Upgrade

The existing structure and basic control apparatus for the irrigation pump system is adequate for the installation of increased capacity pumps. The upgrades required are relatively simple involving motor and pump size changes and a doubling of the fine filter capacity.

A construction cost estimate for installing these features is $87,700.

5.2.7 – River Outfall

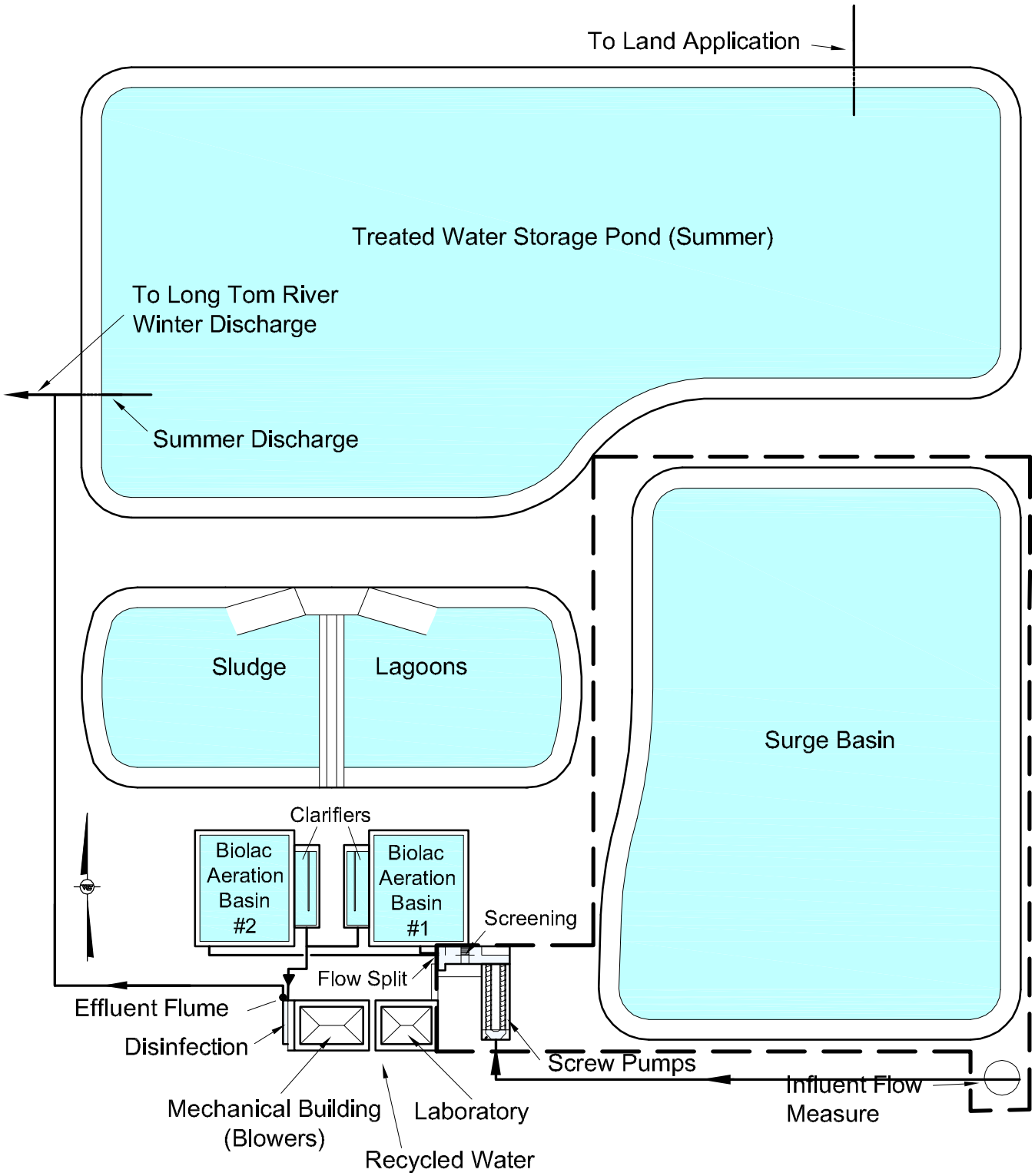
Long term service may require upgrades to the river discharge. These upgrades would require the addition of a diffuser to the outfall. Given the difficulty of this type of construction, we have allocated $37,000 for outfall change, and included these costs in the process design category.

5.3 – Wastewater Treatment Plant Costs

Total upgrade costs are indicated in Table 5.3a.

Table 5.3a

Summary of Treatment Process Costs	
Headworks & Screening	$617,000
Biolac Expansion	$2,083,000
Modify FSL	$736,000
U.V. System Upgrade (2009)	$391,100
Aeration Pipe and W. Hunter Flowmeter	$21,300
Headworks Preliminary Engineering	$58,000
Process Design	$697,200
TOTAL	$4,603,600



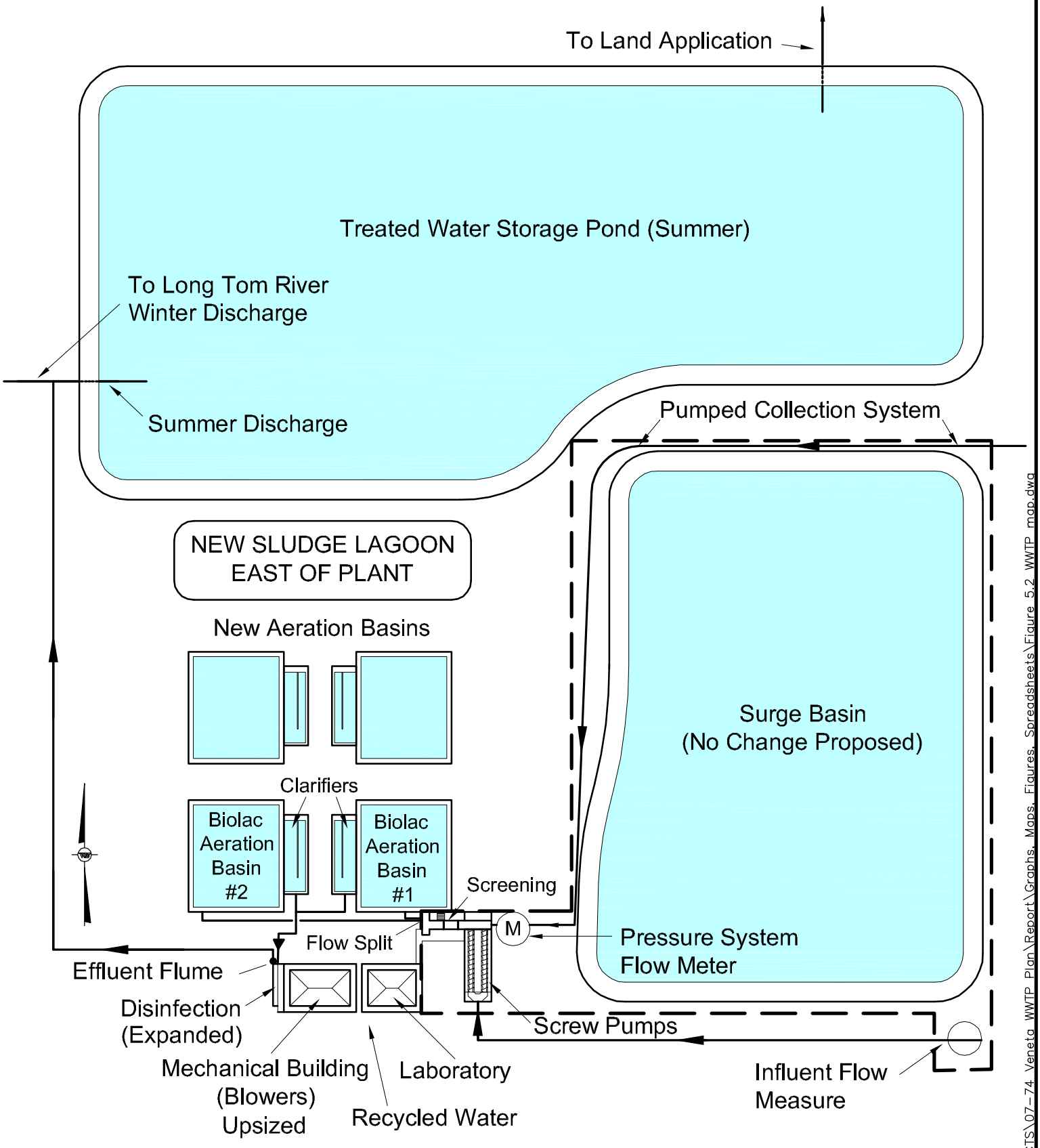
Legend

Headworks

Figure 5.1
Existing Treatment Plant Schematic

WEBER ELLIOTT ENGINEERS, P.C.

3/30/2009 11:39 AM P:\PROJECTS\07-74 Veneta WWTP Plan\Report\Graphs, Maps, Figures, Spreadsheets\Figure 5.1 WWTP map.dwg



Legend

Headworks

Figure 5.2
Future Plant Layout

WEBER ELLIOTT ENGINEERS, P.C.

P:\PROJECTS\07-74 Veneta WWTP Plan\Report\Graphs, Maps, Figures, Spreadsheets\Figure 5.2 WWTP map.dwg 6/30/2009 12:10 PM

6. WATER REUSE

Chapter Summary

This chapter provides the background for the existing water reuse system and reviews water reuse system operational parameters. It examines alternatives for an expanded water reuse. Cost estimates for expanded reuse alternatives are provided. The chapter is organized into sections as follows:

6.1 Background

6.2 Reuse Required

6.3 Reuse Alternatives

6.4 Technology Requirements for Class "A" Reuse

6.5 Reuse Facilities Size and Costs

6.6 Economic Assessment of Reuse and Recommendation

6.7 Recommendation

Section 6.1 establishes water reuse definitions and the current status of the City's system. Section 6.2 explains why water reuse is required as a part of Veneta's treatment regime. Section 6.3 examines two classes of reuse available. Sections 6.4 and 6.5 review the facility costs and size for a selected reuse treatment technology. With costs estimated, the potential for economic recovery of capital invested in expanded technology is reviewed in Section 6.6, and recommendation is provided in 6.7.

6.1 – Background

Treated wastewater effluent reused for beneficial purposes is referred to as "recycled water". The City's existing recycled water system provides water to several treatment plant components for wash-down of equipment and various maintenance duties. The recycled effluent meets the City's discharge permit standards and does not receive additional treatment.

Treated effluent is also recycled when land applied for a beneficial purpose, as with the application to Veneta's poplar plantation and grass area. The intent of the poplar farm was primarily for disposal of summer effluent when river discharge is not permitted. But the production of agricultural crops, such as poplar trees, is also a good example of a "beneficial use" for recycled water. The poplar plantation also serves to sequester carbon and thus mitigates some of the City's carbon footprint.

Veneta's existing drinking water supply comes from a generally shallow aquifer (30 to less than 100 feet deep) that is believed to be inter-connected over much of the City and surrounding environs. Veneta's current poplar and grass application system reduces groundwater demands on the aquifer. As the City grows, more wastewater effluent will be produced and require disposal, greater demand will be placed on the drinking water system, and park areas requiring irrigation will

increase. The increased production of wastewater effluent presents an opportunity for the City to expand its use of recycled water in a way and that may also mitigate demands placed on the City's drinking water system.

In May 2008, the State of Oregon amended the rules for recycled water use to "encourage its use for a variety of beneficial purposes". The state has re-classified wastewater reuse effluent as Class "A" through Class "D", with Class "A" receiving the highest level of treatment. The beneficial uses available for recycled water differ depending on the classification (see Table 6.1a). New reuse options may be feasible that in the past were not.

This chapter examines alternatives for the City's recycled water program. The alternatives are based on two objectives: disposal of wastewater in the summer months and reducing demand on the drinking water system. Different treatment components and infrastructure are required for each objective.

6.2 – Reuse Required

Chapter 4 reviewed regulations relative to the planning, design and operation of Veneta's treatment facilities.

The possibility of an expanded allowance for river discharge into the summer months was reviewed with DEQ. Current DEQ policy is that no allowance would be permitted. DEQ was clear during project discussions, that stream discharge is not and will not be allowed between June 1 and September 30. The primary reason for eliminating municipal discharges from Oregon's waterways is concern about temperature and bacteria on indigenous species. Bacteria can be effectively managed with the U.V. system but the Long Tom River is susceptible to high bacteria counts and additional bacteria loading are problematic to stream health. The stream also tends to be warm and experience exceptionally low stream flows. Protected trout and salmon species (salmonides) generally require stream temperatures below 68 degrees F. to prevent mortality. No temperature discharge data is available from the existing operations but, it is highly unlikely that during summer months the plant could consistently discharge waters at or below 68 degrees F. Use of cooling towers or other technologies for cooling have not been found to be cost effective in other applications.

Table 6.1a

Acceptable Uses For Various Wastewater Treatment Classes					
Beneficial Purpose	**Class A**	**Class B**	**Class C**	**Class D**	**Non-Disinfected**
Irrigation					
Fodder, fiber, seed crops not intended for human ingestion, commercial timber	Yes	Yes	Yes	Yes	Yes
Firewood, ornamental nursery stock, Christmas trees	Yes	Yes	Yes	Yes	No
Sod	Yes	Yes	Yes	Yes	No
Pasture for animals	Yes	Yes	Yes	Yes	No
Processed food crops	Yes	Yes	Yes	No	No
Orchards or vineyards if an irrigation method is used to apply recycled water directly to the soil	Yes	Yes	Yes	No	No
Golf courses, cemeteries, highway medians, industrial or business campuses	Yes	Yes	Yes	No	No
Any agricultural or horticultural use	Yes	No	No	No	No
Parks, playgrounds, school yards, residential landscapes, other landscapes accessible to the public	Yes	No	No	No	No
Industrial, Commercial, or Construction					
Industrial cooling	Yes	Yes	Yes	No	No
Rock crushing, aggregate washing, mixing concrete	Yes	Yes	Yes	No	No
Dust control	Yes	Yes	Yes	No	No
Nonstructural fire fighting using aircraft	Yes	Yes	Yes	No	No
Street sweeping or sanitary sewer flushing	Yes	Yes	Yes	No	No
Stand alone fire suppression systems in commercial and residential buildings	Yes	Yes	No	No	No
Non-residential toilet or urinal flushing, floor drain trap priming	Yes	Yes	No	No	No
Commercial car washing	Yes	No	No	No	No
Fountains when the water is not intended for human consumption	Yes	No	No	No	No

Table 6.1a shows there are two applications where Class B reuse waters differ in application from Class C. That is in the area of stand alone fire suppression systems, and non-residential toilet or urinal flushing and floor trap make up water. Since the City of Veneta has no immediate or significant applications for stand alone fire suppression systems, and no immediate non-residential building use, Class B reuse systems are not reviewed. Class A and Class C technologies are reviewed below.

6.3 – Reuse Alternatives

Two broad based alternatives available to the City for water reuse are examined. Each alternative has various permutations and combinations to the basic description provided. These "sub-alternatives" will be mentioned but not necessarily developed in detail. The reader will recognize that there is considerably more flexibility in the development of a comprehensive reuse program with the advent of the 2008 reuse regulations. For comparison of the reuse alternatives reviewed below we have developed concepts based upon sizing of a reuse plant at a flow of 0.7 MGD. This is the projected average summer flow rate from Veneta's wastewater treatment processes for year 2030.

6.3.1 - Alternative 1: Secondary Effluent Reuse

Chapter 4 noted that the permit limits shown in Tables 4.1a, 4.1b and 4.1c will be retained by the City as its discharge limit. The effluent quality currently produced by the City and the expected effluent quality for an expanded plant (as reviewed in Chapter 5) produces an effluent that meets DEQ's new Class "C" standards. The general discharge standards for Class C effluent are:

- Total Coliforms at 23
- Total Coliforms not to exceed 240 in two consecutive samples
- 70 foot buffer zones from sprinklers
- Signage required.

When designing an agricultural water reuse system at hydrologic agronomic rates it is the engineer's goal to limit the irrigation water applied to the amount of water the plant will adsorb through the roots to meet growth, sustenance and water loss (evapotranspiration).

A significant change that comes from the 2008 reuse regulations that differs from the standards used for planning and construction of the plant in 2001 is that recycle water no longer must be applied at agronomic rates. Under the new rules, allowing for applied wastewater to move past the root zone into the underlying groundwater is acceptable, providing that the groundwater is adequately protected. Thus, water can be land applied at rates exceeding those shown in Tables 5.6a and 5.6b, providing plant health can be maintained and a nuisance condition is not generated.

Nitrogen

The primary concern for most groundwater systems is Nitrogen contamination. The implications for development of a reuse design based upon nitrogen are reviewed

below. The existing pipe and irrigation network has the ability to deliver water to approximately 60 acres (this includes provisions for use of the "Big Gun" mobile irrigation platform). The new regulations no longer require that treated effluents apply at agronomic rates at the crop uptake rate. However, the limiting factor for land application switches from a water application limitation to a nitrogen limitation. The limitation is based upon limiting the amount of applied nitrogen to a crop, to the crop's uptake capacity. Assuming that the treatment plant produces a nitrogen effluent of 22 mg/L it is estimated that year 2030 total nitrogen loading will be about 16,000 pounds. Table 6.3a indicates the existing irrigation application area has the capacity to handle up to 18,400 pounds of nitrogen per year (June through September application). This is a fairly typical nitrogen level for an activated sludge plant. However, the Biolac system can operate in what is called a nitrification/denitrification mode without the addition of equipment or operating expense. When operated in the nitrification/denitrification mode a plant effluent of 10 mg/L, or less, of nitrogen would be expected, resulting in total annual Nitrogen output of about 7,300 pounds.

Table 6.3a

Existing Land Application Area			
Element	**Uptake Rate (lbs N/year/acre)**	**# of Acres**	**Uptake (lbs/year)**
Poplars	260	18.7	4862
Grasses	220	61.6	13,552
Buffers	N.A.	20	N.A.
	Total Uptake Capacity *		18,400

*Capacity Based on Nitrogen Loading)

Some isolated nitrogen data has been made available. Results of Nitrogen sampling from October 2007, June 2008 and October 2008 are summarized in Table 6.3b.

Table 6.3b

Nitrogen Sampling Results				
Date	**Total Kjeldahl Nitrogen Mg/L**	**Ammonia Nitrogen**	**Nitrate Nitrogen**	**Total**
10/08	2.2	0.2	1.4	3.8
6/08	8.5	6.3	11	25.8
10/07	1.7	0.35	1.4	3.4

These values indicate that the assumption above may be reasonably conservative for planning for future nitrogen loading. We continue to recommend that effluent sampling for Nitrogen be conducted quarterly and that the operator run the plant in

a nitrifying/denitrifying mode to determine nitrogen removal performance under various modes of operation.

Based upon the current effluent receiving area and estimates of nitrogen loading the existing land application area is of sufficient size that the City can produce a Class C effluent and not require additional land.

Soil Uptake Capacity

The limiting factor for land application of recycled water can become the ability of the soil to take water. The Lane County Soil Survey notes that the predominant soil in the reuse application area is McBee silty clay loam. This soil typically has a 24 inch silty clay loam surface with a mottled silt loam underneath for about 17 inches. The substratum typically goes to 60 inches or more and is a dark brown, mottled silt loam. This soil has a water capacity of 10 to 12 inches (the water capacity is the amount of water the soil can deliver to a crop).

The soil has a “water supplying capacity” of 18 to 24 inches. This means the soil holds 6 to 12 inches of water within the soil structure in the spring before crops begin to take the available 10 to 12 inches of water. The McBee soil has an estimated permeability of 0.6 to 2.0 inches per hour and has a shallow water table of 2 to 3 feet from November through April. Land application design manuals suggest that the water application rate not exceed four percent of the soil permeability. Thus, for a soil with permeability rate of 0.6 inches per hour the maximum design permeability rate would be 0.024 inches per hour (17.28 inches per month). A water reuse system designed for maximum water uptake should not exceed the 17.28 inches per month or 70 inches over the typical 122 day water reuse application period.

An application at 70 inches of wastewater from June 1 through September 30 would maintain saturated soil conditions over the application area. At an application rate of 70 inches it is estimated that the total nitrogen load on the system would be 360 pounds per acre. This would exceed nitrogen uptake rates for poplars by as much as 100 pounds per acre and grasses by 140 pounds per acre and result in a deep percolation of effluent estimated at 12 mg/L Nitrogen. This exceeds the drinking water standard for nitrogen and would not be acceptable. Nitrogen loading becomes the limiting factor and the land application system must be designed based upon potential nitrogen loads to the groundwater. The values shown in Table 6.3a serve as a guide to land application effluent limits. The practical application limit based upon Nitrogen application is calculated to be 58 inches for poplars and 41 inches for grasses.

Based upon the estimated 16,000 pounds of Nitrogen and the grass uptake rate described above and a future (year 2030) summer treatment plant effluent of 0.7 MGD it is estimated that 72 acres of grass is required to meet the land application requirements. However, at these high application rates the City would experience difficulty managing equipment on saturated soils, the grass would be prone to disease and harvest could not be efficiently managed. A larger area (estimated 30 percent at a minimum) must be employed to provide adequate flexibility for

management and harvest. As an alternative, some land could be retained in poplar production to increase nitrogen uptake and balance water use.

It is recognized that the City is frustrated with the cost of poplar plantation management and would prefer to grow grass and employ local contractors for harvest and management. But, based upon this analysis the City will require the higher Nitrogen uptake capability of the poplar trees in order to effectively manage the reuse system. The City must weigh the management burden of additional grass acreage versus management of tree stands. Part of the answer to this dilemma is reviewed in section 6.5. If the City determines that it will proceed with the development of a reuse system then management of the reuse area for grass will be satisfactory. If a reuse system is not expanded for inter-city uses then a portion of the poplar plantation must be retained for Nitrogen control or the land application area expanded.

6.3.2 - Alternative 2: Class "A" Reuse

The new Class "A" reuse standard has the following criteria:

- Total Coliform at 2.2 per 100 mL
- Total Coliforms not to exceed 23 in two consecutive samples
- Monitored turbidity of less than 2 NTU's
- Water is filtered
- Water may not be sprayed into an area where food is being prepared (instead of a buffer zone requirement)
- Signage required but wording may be negotiated with the Department

As indicated in Table 6.1a, Class "A" waters can be reused for agriculture, landscape, parks, playgrounds, school yards, residential landscapes or other areas assessable to the public. They can also be used for car washing, decorative fountains, recreational impoundments, and even groundwater recharge providing the rules established by the Department of Water Resources are followed. The regulations go on to say that Class "A" recycled waster may be used for "Any beneficial purpose authorized in writing by the department pursuant to OAR 340-055-0016(6)." Clearly this opens up a new world for reuse in the State of Oregon.

Water quality requirements for industrial reuse vary. Some industries require very high quality water while others may need only minimal levels of treatment. However, in order to ensure adequate protection of public health all regulatory agencies do require a certain level of disinfection prior to reuse. Veneta does not currently have an industry that would use significant quantities of reclaimed water. In addition the City, and the industry, would need to invest in the infrastructure to transport the treated water to the industry's point of use.

Non-potable private use is the use of reclaimed water for non drinking non-direct human contact water functions. Examples of non-potable private use would be the use of reclaimed water for lawn watering and landscape maintenance and even to the point of installation of distinct plumbing systems that would use the reclaimed

water for refilling of toilets or other non-human contact uses. Currently Oregon plumbing codes do not allow for the use of reclaimed water within the home. Negotiations with the Oregon DEQ and the Oregon Health Division would be required to use recycled water for building plumbing. DEQ officials and legislators continue to work toward a resolution to the recycle limitation.

Non-potable public use is really identical to private use except that the application is to a public building. The implication here is that the public building may have more control and limits to indiscriminate access and reduce the risk of inadvertent exposure to the treated wastewater. This level of reuse is being used in some locations within the United States.

Aquifer recharge involves the re-injection of treated wastewater into the groundwater. Technologies exist such that the treated effluent can consistently meet and exceed drinking water standards. In order to provide an even higher level of public protection for viral re-growth, aquifer recharge reuse systems are designed such that the introduced waters have extensive opportunity to mix with the groundwater prior to extraction. Aquifer recharge programs are currently being operated in Los Angeles and Orange counties California, as well as in the Northeast. . Elements of the California recharge programs were installed in the 1970's and enhanced treatment processes were added in 1996. One of the premier examples of water reuse is found in Orange County, where a series of treatment processes are employed prior to use of the treated water as groundwater recharge into a drinking water aquifer. To see an overview and guiding principles of this treatment and use arrangement go www.gwrsystem.com on the web, or Google wastewater recycling.

6.4 – Technology Requirements for Class A Reuse

Chapter 5 established that the existing Biolac technology is suitable for continued agriculture reuse on poplars, grasses and other non-human consumption crops. The treatment plant upgrades identified in Chapter 5 would be required to consistently create an effluent of Class C quality. However, as explained, the City will be required to invest as early as 2017 in filtering technology to meet the BOD_5 and TSS stream discharge limitations. These plant upgrades will create a tertiary (filtered) effluent that will meet Class "A" standards excepting standards relative to disinfection. The development of a Class "A" effluent becomes a plausible and an economic public benefit when reuse waters can be developed and distributed at a cost equal to or less than the cost of development and distribution of recycled waters.

The production of a Class "A" effluent for water recycle requires enhanced disinfection, increased monitoring and filtration to comply with the regulations. However, DEQ has recognized a relative new technology called Membrane Bioreactors (MBR) which uses a small (generally about 2 microns) tube as the water withdrawal mechanism. It is this small pore size that serves as the filtration requirement for MBR effluents. Obviously it takes a tremendous amount of small tubes to manage flows of 0.7 MGD.

The advantages of a MBR plant is that these plants require a small footprint, they are operationally reliable and simple (much like the Biolac system), produce an excellent effluent and can easily manage odors. It is outside the scope of this project to weigh the merits and costs of MBR against Biolac but such an assessment would be required should the City prepare a new facilities plan for scheduled plant upgrades. If the City were to continue to use a Biolac system DEQ may allow authorization of plant expansion using the existing approved facilities plan. We would recommend that MBR technology be examined as an alternative to the Biolac system in a preliminary design report as part of the 2013 plant expansion – especially if the City would decide to pursue the development of a Class "A" effluent.

6.5 – Reuse Facilities Size and Costs

Section 5.3 provides a review and discussion of the requirements to construct a tertiary wastewater treatment plant. A tertiary wastewater plant described here provides a filtered effluent with BOD_5 at TSS below 5. Adjunct facilities to upgrade these plant processes to Class "A" are reviewed below.

6.5.1 - Processes

The size and process requirements for a Class "A" facility are developed based upon the existence of a 0.7 MGD tertiary plant constructed in 2017 as part of the facilities requirements outlined in Chapter. 5. The Class "A" processes required to provide the upgrades from a tertiary plant to a Class "A" facility is:

- Enhanced operator training and certification
- Continuous monitoring of turbidity
- Expanded U.V. capacity
- Supplemental chlorination to provide system disinfection residual

6.5.2 - Distribution

There is no compelling reason to treat waste water to Class "A" standards and then apply to the poplar plantation where Class "A" standards are not required. Figure 6.1 shows a schematic of a basic water reuse distribution grid that delivers water to four locations suitable for immediate use (Territorial Sports Fields, Territorial Park [Skatepark], WestLane Greenway and City park). These areas have a combined land area of approximately 21 acres. Typical park watering rates in western Oregon will range from 2.2 feet to 3.0 feet of water per summer. At an application rate of three feet, the combined capacity of these areas to accept Class "A" water is 63 acre feet of water.

In addition to the public use areas already identified, the City's Urban Renewal agency is pursuing a development between Broadway and Waldo and west of Territorial Highway. This development is scheduled for construction in 2009 and may have reuse potential for both landscape maintenance and public reuse. In Figure 6.1 we have assumed a demand of 100 gpm for this area and perhaps as much as three acre feet of water use during summer. Based upon 2030 design

estimates the total summer demand for the 21 acres of reuse and the urban renewal development is estimated at 66 acre feet. This total demand (21 million gallons) represents about 30 days of wastewater effluent or about 25 percent of the summer plant production.

6.5.3 - Costs

Costs presented here follow the general guidelines for costs reviewed in section 3.8.1. The reader is advised however, that the technologies applicable to disinfection using U.V. systems, turbidity monitoring and even supplemental chlorination continue to experience rapid changes. Costs estimates provided herein may undergo significant fluctuations either up or down due to regulatory and technology changes.

We estimate that turbidity monitoring technology would include continuous monitoring and redundant systems. An appropriate laboratory turbidimeter currently costs about $3,000. Installation of redundant units plus training and field installation is estimated at $21,000.

Supplemental chlorination involves the addition of liquid sodium hypochlorite to the process stream along with associated storage facilities and chemical feed pumps. We recently (2008) had an opportunity to specify a continuous recording chlorine monitoring unit for a drinking water facility. The basic unit costs just under $4,000. Again redundant units would be recommended and installation costs and training would bring the installed costs for complete operational system to an estimated $32,000. The City many want to consider on site generation systems for chlorination to meet sustainability goals.

Expansion requirements for the U.V. system would be the most significant. The U.V. system installed for the scheduled 2017 plant would have the hydraulic capacity to manage the flow, but significant U.V. lamp additions would be required. Lamp and installation costs for these U.V. upgrades are estimated at $268,200.

A basic distribution system for a Class “A” effluent is shown in Figure 6.1. It is estimated that this distribution system would cost $460,000. Table 6.5a provides a total costs for developing a Class “A” system from a tertiary plant. The costs shown include appropriate planning level engineering and contingencies.

Table 6.5a

CLASS “A” ESTIMATE OF COST				
City of Veneta - Wastewater Facility Planning				
Item	**Quantity**	**Unit**	**Price per unit**	**Total**
Turbidimeter	2	EA	$ 10,500	$21,000
Chlorination System	1	L.S.	$32,000	$32,000
U.V. Upgrade / Changes	1	EA	$268,200	$268,200
Distribution Network	1	L.S.	$460,000	$460,000
Total				**$781,000**

6.5.4 - Class "A" Technology – Operation and Maintenance

In addition to the facilities required for a Class "A" treatment plant, the regulations specify unique operation and maintenance requirements. These requirements include increases in operator attention, added responsibility for monitoring and reporting, expanded laboratory utilization and increased energy consumption. Table 6.5b provides an estimate of annual increased costs to produce and distribute a Class "A" effluent. We note that it does not include an allocation of costs for maintenance of park facility sprinkler heads and controls at the point of receiving the water.

The operator of a Class "A" facility will need to have expanded education and experience to manage the requirements of a Class "A" facility. Expanded training in laboratory analysis, sampling, reporting and documentation is required. An operator licensed and capable of operating a Class "A" plant will demand a higher compensation.

Currently the City sends out weekly Total Coliform samples to an outside laboratory. Because the Class "A" criteria requires a daily total Coliform sample it would likely be that the Coliform samples would be processed in the laboratory at the existing plant. With the added responsibility of operating a Class "A" plant, the City operations would need to dedicate a full time operator to the treatment plant.

The Class "A" treatment plant and distribution system outlined above will require increased energy consumption. It is estimated that the pumps required to lift treated wastewater into the filters will require 2.8 continuous horsepower. The power consumption for these pumps would require an estimate $500 per summer. In addition re-pumping of 46 acre feet of water to appropriate pressure for the distribution system is estimated to cost an additional $860.

The continuous up flow filter requires a compressor to move the sand through the backwash operation of the filter. This is sized as a 5 horsepower compressor and is estimated to cost $300 per summer.

A Class "A" facility is required to continuously monitor turbidity and provide daily assessment of Total Coliforms. The turbidity measurement can be made by the use of a constant recording turbidity metering device. Turbidity records can be recorded and downloaded to a computer file with an output for alarms. We estimate $1500 per year for maintenance on the turbidity meter.

Table 6.5b

CLASS "A" ESTIMATE OF ADDED ANNUAL OPERATION COST			
City of Veneta - Wastewater Facility Planning			
Item	**Quantity**	**Unit**	**Total**
Operator Training	To Full Time	EA	$21,000
Laboratory Supplies	1	L.S.	$2,000
Energy	46	AC. FT	$1,500
U.V. Operator	1	L.S.	$2,000
Special Maintenance	1	L.S.	$1,500
Total			**$28,000**

6.6 – Economic Assessment of Reuse

An economic assessment of Class "A "reuse requires a compilation of capital for the processes, capital costs for the delivery infrastructure and a comparison the expanded operation and maintenance costs against the continued application of a Class "C" effluent for agriculture uses. Table 6.5a provides an estimate of the capital costs to upgrade a tertiary plant of 0.7 MGD to a Class "A" plant and a basic distribution system. Table 6.5b estimates increases in operation costs for such a facility. This section compares this capital and annual investment to an investment in other water resource investment alternatives.

6.6.1 – Water Resource Development Costs

Veneta's historic water supply is a narrow aquifer found under most of the City. The aquifer is as shallow as 20 feet and as narrow as 25 feet in the northwest segments of the City and has been found to be as deep as 100 feet in some isolated areas near the east end of town.

Water supply is a major long-term issue of concern for Veneta, as it is with most communities in the state. Innovative, farsighted solutions are necessary to meet the challenges of providing water for growing populations. More and more utilities are exploring technologies and sources that, in the past, would have been overlooked.

Water source investigations conducted since 2000 by the City have failed to find portions of this aquifer that are sufficiently deep and contain sufficiently permeable and yielding formations to ensure economic resource development. Though the aquifer has been found to be reasonably ubiquitous, its yields are costly to develop. An alternative supply is the use of the Long Tom River and/or the waters of Fern Ridge Reservoir. The Long Tom River is severely impacted by upstream land use practices which impact the water quality. The reservoir backwaters are loaded with suspended solids and petroleum combustion products associated with recreation activity on the lake. As stated in the water master plan, use of these sources brings the city under a completely new set of management regulations know as the

Surface Water Treatment Rule (SWTR). The SWTR has increased monitoring requirements.

The City's most recent (SWTR) water resource development project involved the development of Wells 10 and 11: bringing them on line and into the water delivery system. This facility was added into the City's water supply system from 2006 through 2008 and involved the development of two wells and the construction of a 400 gpm capacity iron removal plant. The total cost of the plant including planning, wells, construction, materials and equipment was approximately $1.8 million. Well 10 has a capacity of about 180 gpm and well 11 has a capacity of 100 gpm. Firm capacity for this complex is estimated at 280 gpm. 280 gpm yields a gross production capacity of less than 1.2 acre feet per day (383,000 gallons). This represents 143 acre feet through a typical 122 day summer (June 1 through September 30). We do not have available additional operator and maintenance costs for the Well 10 and 11 complexes.

The use of recycled wastewater effluent to irrigate 21 acres during a hot summer week would have the reuse system supply one inch of water on the 21 acres. This one inch of water would be about 570,000 gallons or equate to taking 81,000 gallons per day of demand off of the potable water system.

One means of comparing different development costs of groundwater and reuse Class "A" supplies is to examine costs for a compatible yield. The $1.8 million expended on the well 10 and 11 complex has the ability to produce 143 acre feet of water over a 122 day summer use period. This is a capital cost of $12,600 per summer acre foot water yield. The water reuse project defined in this section expends an estimated $781,000 to provide 66 acre feet of water or $11,800 per summer acre foot water yield.

Based upon peak capacity alone the paragraph above indicates that the development of a 66 acre foot capacity water reuse system is approximately equal to the further development of groundwater supplies. These costs look only at the experience with wells 10 and 11 which is recent and applicable experience but does not account for increased potable water system capacity infrastructure such as valves, reservoirs, controls and other accessories required to make a whole, functional and integrated system. Future water development cost may significant exceed the cost for development of the Well 10 and 11 complex. It is also important to note that with the expenditure of nearly $800,000 in water reuse infrastructure the City is poised to expand that system that would be capable of handling perhaps as much as 130 acre feet of distribution.

6.7 – Recommendation

This analysis shows that a current investment in Class "A" technology and a water reuse distribution system may be cost effective. But in addition to the strictly economic analysis the development of the Class "A" infrastructure does add value to the community. The addition of a Class "A" water reuse system would allow the City to maintain parks and recreation areas in a lush and green manner adding aesthetic appeal to the City's facilities and contributing to a sense of community pride.

The recommendation coming from this analysis is that the City should continue to monitor and assess the cost of water resource development and compare that costs against new and improved treatment technologies. We recommend that a comprehensive re-evaluation of the cost and environmental suitability of Class “A” reuse be undertaken as part of the assessment and plant expansion scheduled for year 2017. In addition, the City can continue to monitor State and Federal grant opportunities available for water reuse systems that would tip the scales toward economic development of water recycle infrastructure.

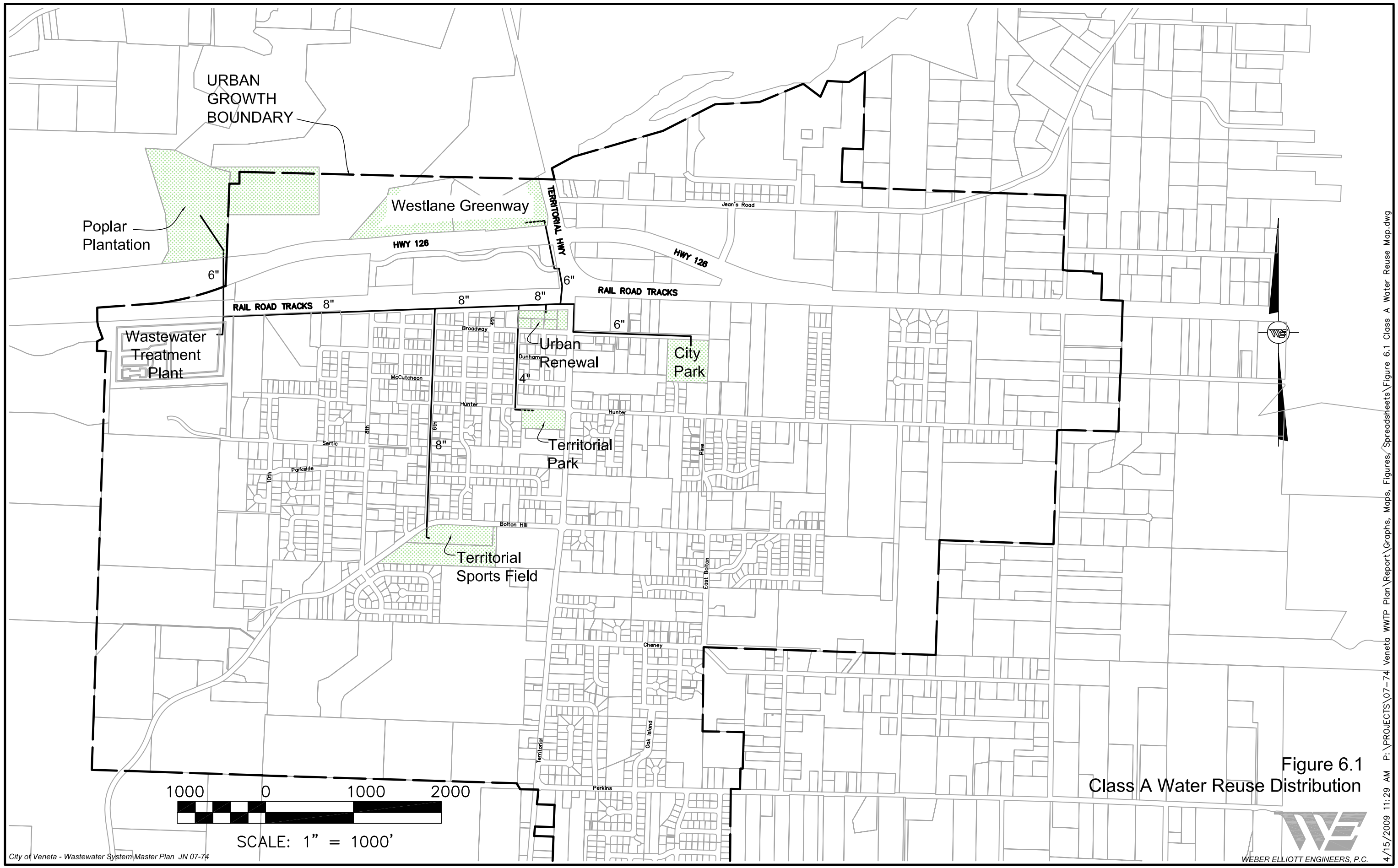


Figure 6.1
Class A Water Reuse Distribution

City of Veneta - Wastewater System Master Plan JN 07-74

WEBER ELLIOTT ENGINEERS, P.C.

4/15/2009 11:29 AM P:\PROJECTS\07-74 Veneta WWTP Plan\Report\Graphs, Maps, Figures, Spreadsheets\Figure 6.1 Class A Water Reuse Map.dwg

7. CAPITAL IMPROVEMENT PLAN

Chapter Summary

This chapter integrates the selected collection system (Chapter 3), treatment system (Chapter 5) and Water Reuse (Chapter 6) alternatives into a single schedule. The schedule will allow the City to plan, budget and implement the segments of the plan as the implementation elements come due.

Each capital project is provided a number. Collection system improvement programs are designated with a “C”, treatment projects with a “T” and water reuse program elements with an “R”.

7.1 – Capital Improvement Plan

The recommended capital improvements are described in chapters 3, 5, and 6. The Capital Improvement Program is summarized in Table 7.1. The costs shown in the table represent total estimates of costs and include construction, engineering, contingency and administrative costs. Generally each project is spread over a two year period with a recommendation that the engineering be authorized in the initial year with construction during the following year. Please refer to Figure 7.1 for the specific locations of each of the projects specified below.

7.1.1 – Collection System Elements

The selected Collection System plan is Alternative 1 as presented in chapter 3. The implementation schedule of Alternative 1 is phased over 13 distinct projects shown on the attached capital program (Table 7.1). Each capital project is provided a number. The number and short description corresponds to a capital project identified for the collection system in Chapter 3.

The discussion below provides additional detail relative to schedule and budget on a project by project basis. Collection system projects are presented in an order that follows the progression of the system bypass development and not necessarily in a chronological order. Most of the collection system work is required to be completed by 2018.

Project C1 – 2009:

This project is scheduled to begin to address the immediate need to correct the system overflows and the bottle neck conditions experienced at the intersection of Territorial and Hunter. The first element of the project is the addition of a Chimney at the Treatment Plant (Manhole 301). This will alleviate flooding and overflow when the screw pumps are unable to manage peak flow conditions. A one foot manhole rim extension is required. This is a minor project that City crews could complete. It amounts to ordering a manhole extension, removing the existing rim and lid, grouting the extension in place and re-installing the rim and lid.

The second element of Project 1 involves construction of the relief sewers at the intersection of Territorial and Hunter. The project will involve rebuilding of two

manholes and the installation of a third manhole south of the intersection. One of the reasons this intersection contributes to backup of sewer flow is that all of the manholes intersect at 90 degrees. Although this is the customary design approach the presence of multiple lines and minimum slope contributes to significant hydraulic inefficiencies at this intersection. The design concept prepared for the C.I.P involves setting a new manhole approximately 70 feet south of the intersection and construction of 21 inch diameter pipe that routes the sewer in an arc by deflection of the pipe joints. The arc would alter the wastewater direction from north to the west ensuring a smoother transition to Hunter Road. This is an expensive project that will require extensive coordination with ODOT for access to the Highway intersection. If the budget will allow, design can be initiated in spring 2009 and bid and construction in late summer of 2009.

Project C2 – 2010:

This project provides for correction of the flow capacity issues relative to undersized and minimal slope pipe in Territorial highway. 2,290 feet of eight-inch concrete piping under Territorial Highway will be abandoned and/or replaced with 10, 12 or 15-inch polyvinyl chloride (PVC) gravity pipe. The project will include an evaluation of pipe bursting suitability and selection of the least cost method of replacement. 70 service laterals would need to be re-connected to new sewer pipe. 12 new manholes would need to be installed with new pipe. The recommendation is design in 2009 and construction in summer 2010. It is also recommended that the 2010 budget include sufficient funds to begin alignment studies for Project C3.

Project C3 – 2013:

This project initiates the beginning of the collection system bypass system. It involves design and construction of pressure pipe and associated infrastructure for the pipe between the wastewater plant and Territorial Highway. This project would include alignment studies and design for connection to the existing discharge from the WestLane pump station. This would connect the WestLane pump station to begin to use this line and WestLane flow would be removed from the Territorial/Hunter intersection. Disconnect the force main for WestLane pump station and re-route and reconnect the discharge into the new pressure network. 500 feet of force main would be abandoned. This project is scheduled for design in winter 2012 and construction in 2013. Note that this design must be made concurrent with project T2 to ensure that the wastewater facility headworks are ready to receive the flow modifications.

Project C4 – 2017:

This project extends the pressure bypass sewer from the termination described in Project C3 to a pressure line connection near the current intersection of Hope Lane with Highway 126. The driving force relative to scheduling for this project will be the development pace, or interest, that occurs on the east and north end of Veneta. This project is a precursor to providing the pipe infrastructure for Projects C5 through C12. It will capitalize on the alignment studies completed in 2010 as part of Project C2. Project 4 will install 12 inch pressure pipe along the selected alignment. It is scheduled for design in fall of 2017 and construction in summer 2018.

Project C5 – 2021:

This project is the final leg of the 12 inch diameter pressure bypass sewer that provides for connection at Hunter Road. It is scheduled for design in winter of 2021 and construction in 2022. If significant development occurs in the east area of the City this project may need to be accelerated to ensure that adequate flow bypasses the wastewater influent pumps.

Project C6 – 2017:

This project initiates the expansion and development of the northeast area. It includes construction of a new pump station north of Jean's Road (about 150 feet south of the urban growth boundary), installation of duplex 10-horsepower pumps and 2,370 feet of four-inch diameter force main. The force main will connect with the 12 inch pipeline constructed as part of Project C4. One of the elements of Project C6 is the establishment of City criteria and standards for private pumps for service into pressure sewer systems. Private pressure sewer systems are scheduled for a high percentage of the northeast area and installation, maintenance and public acceptance will be easier if clear standards are established. It is recommended that the City establish these standards as early as 2010 so that they are in-place for implementation of Project C6 and other Projects requiring individual pumps for sewer service. Project C6 is scheduled for design in 2017 and construction in 2018.

Project C7 – 2015:

This project provides for construction of the east side elements. Project C7 is the design and construction of a new pump station on Huston Road 800 feet South of Hunter Avenue and 4,920 feet of 8 inch diameter force main that connects to manhole number 604 on Hunter. Manhole 604 allows for the wastewater to proceed to the Pine Street lift station. The pump station is schedule to include dual 50-horsepower pumps. The force main would extend from the lift station to approximate site for connection into Project C5. Since Project C5 is not scheduled until 2017 this pressure line will temporarily go into the Pine Street lift station until project C5 is complete. Project C7 also includes the influent gravity pipe that will serve the east part of the City. The gravity pipe included in this project includes 12 inch pipe from the intersection of Hunter and Huston and 8 inch pipe in Huston Road to Tidball. All of the other gravity pipe scheduled in Project C7 is 8-inch diameter and will be incorporated into development schedules as parcels are developed. This project is scheduled for design in 2015 and construction in 2016. Accelerated development pressure in the east portion of the City may require that Project C7 be initiated sooner.

Project C8 – 2021:

When the Headworks are installed as part of project T2 or when collection system capacity issues begin to occur on the west Hunter collection line then Project C8 will need to be implemented. The model estimates that this project will be required in 2021. It will involve the construction of a force main extension from the east side lift station into the pressure system installed as part of Project C5.

Project C9 – 2021:

This project provides for the extension of a 12 inch force main from the Pine Street lift station for connection into the line installed as part of Project C5. The "trigger" for this project will be capacity issues on the influent pump station (Project T2) or insufficient capacity for the West Hunter trunk line. The system modeling conducted with this report suggests that the implementation of Project C9 will occur concurrent with the implementation of Project C8. It is listed as a distinct project because if significant development growth occurs in the Pine Street lift station service area it may be necessary to implement the requirements of Project C9 separate for Project C8.

Project C10 – 2013:

At such time that expanded development and use of the collection system occurs in the WestLane area the existing pump at the Westland pump station will need to be upgraded. Project C10 provides for replacement of the pumps. The recommendation is that design occur in 2013 and construction in summer 2014.

Project C11 – 2013:

This project initiates the beginning of the collection system bypass system. It involves design and construction of pressure pipe and affiliated infrastructure for the pipe between the wastewater plant and Territorial Highway.

Project C12 – 2012:

Project C12 provides for installation of an integrated control network for monitoring and control of all of the wastewater collection mechanical and electrical elements. It provides the design and communication infrastructure to coordinate control functions for the existing and proposed lift stations. This project is scheduled for implementation in 2012 so that the control network can be fully integrated into the treatment plant headworks changes (Project T2) and that the control functions can be put in place prior to design and construction of the North and East lift stations. Initial control studies should be initiated to determine if the control system will be operated from the Public Works Yard (on Broadway) or from the wastewater treatment plant. These initial control studies will also define if the selected control communication system will be by use of a wired system installed concurrent with the pressure pipeline projects or by use of radio controls.

Project C13 – 2029

Project C13 consists of three projects that are integrated into the capital improvement plan because the computer collection model identifies these locations have having service but may or may not require implementation. A coordinated maintenance log and an understanding of the hydraulic problems experienced in these areas will serve as triggers for project initiation. The project elements are:

- ***8th Street Gravity Branch*** – The eight-inch gravity pipe under Parkside and under 8th Street in the Bowling Green development will be upsized to 10 or 12-inch PVC pipe to meet 2030 peak load requirements. Although the model

clearly identifies these pipes as a problem area it may be possible with an aggressive maintenance program to delay this project for the life of the C.I.P.

- ***Oak Island Gravity Branch*** – Like the 8th Street changes in the Bowling Green subdivision this project may not be required. It provides for gravity pipe under Cheney and under Oak Island to be upsized to 10-inch PVC pipe to meet 2030 peak load requirements. The project is scheduled for 2029 but may be required earlier.
- ***New East Sewer Service*** – The north eastern sections of Veneta (north of Highway 126 and east of Cornerstone are scheduled for service using privately maintained pumps at the property. Project 6 alludes to the development of standards for this service. This project element is included to provide for public supported infrastructure needs.

7.1.2 – Treatment Process Elements

This section reviews the projects identified in Chapter 5 for wastewater treatment. Each project is given a designation title beginning with the letter "T" to signify that it is a treatment related C.I.P. As shown in Table 7.1 the treatment projects are initiated in 2009 and continue through 2018.

Project T1 - 2009:

Project T1 has two elements: Air leakage repair and flow measurement. The first project elements calls for the immediate repair of the air leakage experienced at the plant. The air leakage is contributing to significant energy loss at the plant and has the potential to impact wastewater effluent quality during periods of high organic load because the treatment basins will not be able to deliver sufficient oxygen to the activated sludge.

The second project element is the installation of a flow meter station at the intersection of Hunter and 10th. This flow meter station was scheduled to be added to the flow measurement in the DMR's in 2006. Completion of this project should not be delayed.

Both elements of Project T1 should be authorized for design immediately. Additional research will be required to recommend the most cost effective solution for the air leaks but it should be possible to have the project completed by third quarter of 2009. The specifications and installation for the flow measurement components can be completed in the second quarter of 2009.

Project T2 - 2010:

Project T2 involves the preliminary engineering to establish how projects C3 and the remaining treatment project will be implemented from a design standpoint. It involves sufficient preliminary engineering to establish grade lines for incoming wastewater, screens and downstream treatment processes. This project helps to ensure that all of the treatment projects are integrated into a common plan and use the same planning and grading criteria.

Project T3 - 2011:

Project T3 involves two elements of change to the plant headworks: modification of the facilities to accommodate increases in flow rate over the planning period and changes required in the screening that prepare the plant headworks for receiving of the collection system changes identified in Project C3. As currently scheduled Project T3 would enter design phase in 2011 with construction scheduled in 2012. The flowrate accommodations and the screening changes will need to be done as a single project to ensure full integration of the headworks for future collection and treatment system management.

Project T4 - 2011:

Project T3 serves as a preparatory project for Project T4. It involves the abandonment of the existing facultative sludge lagoons and the construction of new lagoons east of the existing plant. This project needs to be completed in order to make allowance for space for the expanded aeration basins. It is possible to combine this project with project T4. Project T3 is scheduled for design in 2011 and construction in 2012. We would recommend to the City that in conjunction with the design of Project T3 that the groundwork for project T4 be outlined for City and DEQ review. This will mean that a preliminary design report for Project T4 is developed in 2011 as well.

Project T5 - 2012:

Project T4 provides for the design and installation of the expanded biolac aeration and treatment processes. It is the most capital intensive of the C.I.P. projects on the list. The project schedule calls for design initiation in 2012 but depending on plant loading and development the construction of the upgrades may not occur until 2013 or even 2014.

Project T6 - 2010:

As stated in Chapter 5 the existing U.V. system is at capacity. The integration of the U.V. system is critical for future plant additions and water reuse elements. An understanding of the overall fit of these systems means that Project T6 should be authorized in late 2009 (allowing time for review and approval of the C.I.P. and master plan) and facilities installed as soon as design is complete. Because much of the U.V. expansion is not weather dependent this work can be completed outside the normal summer construction season.

Project T7 - 2018:

Project T7 involves laying the basic infrastructure for the construction of needed water system upgrade to ensure permit compliance after existing plant capacity is reached (estimated 2014).

7.1.3 – Water Reuse Project Elements

Water reuse projects incorporated into the C.I.P. program are shown in Table 7.1 and given project designation beginning with the letter R. Brief descriptions are provided below.

Project R1 - 2010:

Project R1 involves a forest health assessment of the existing poplar plantation and an assessment of optimal crop rotation and harvest. Although it is a minor project it establishes the criteria and possible phasing for project R2. And it will provide more realistic budget numbers for management of the poplar plantation.

Project R2 - 2011:

Project R2 involves crop harvest. Oregon forest management statutes, reviewed in Chapter 4, in the past required that silviculture land that is managed for periods longer than 12 years must have an environmental impact statement. Exceptions to those requirements have been itemized in 2008 amendments to the rules. Project R2 provides for poplar harvest and replanting as indicated in Table 7.1. It is recommended that 2011 be used to establish planning and harvest criteria and the trees be harvested in 2012.

Project R3 - 2015:

Project R3 involves the pump station modification required to accommodate changes in the reuse system. The implementation of this project needs to occur very soon after the plant expansion scheduled as project T5. As currently scheduled Project R3 would enter design phase in 2015 with implementation of the changes in 2016.

Project R4 – 2020:

Project R4 provides for comprehensive upgrades to the U.V. system to meet higher reuse standards. These higher standards will be required as the effluent application of Class A waters within the City.

Project R5 – 2017:

Project R5 provides for application of the Class A waters developed as part of projects T7 and R4 to areas within the City which will allow for direct public contact. This project provides for a basic water reuse distribution system that serves reuse water to the areas shown in Figure 6.1. As recommended in Chapter 6 a feasibility study requiring and estimated $50,000 should be initiated in 2017. If the project is deemed feasible at that time then construction of project elements would most likely begin in 2019 and be phased in over the next two to three years.

CAPITAL IMPROVEMENT PLAN

Project*		2010	2012	2014	2016	2018	2020	2022	2024	2026	2028	2030
	Collection System											
C1	Upsize to new 21" modified gravity pipe under Hunter/Territorial intersection	$282,800										
C2	Upsize to new 10", 12", 15" gravity pipe under Territorial south of Hunter from MH I-5 to MH Ba-6	$542,300										
C3	Install new 12" pressure line from Waste Water Treatment Facility to Westlane forcemain		$510,500									
C4	Install new 12" pressure line from Westlane forcemain to Hwy 126 near railroad					$154,300						
C5	Install new 12" pressure line from Hwy 126 near railroad to Hunter							$148,900				
C6	New North Pump Station north of Todd Way near shore of Fern Ridge Resevoir, includes 4" forcemain					$559,800						
C7	New East Pump Station 800 feet south of Hunter off Huston, includes new 8" forcemain and 8" or 12" gravity				$1,356,800							
C8	Install new 8" forcemain extension for New East Pump Station							$39,400				
C9	Install new 12" forcemain extension for Pine Street Pump Station							$34,700				
C10	Replace pumps at Westlane station			$32,600								
C11	Replace pumps at Pine Street station				$59,700							
C12	Main Pump Control Center (SCADA Brain)		$46,400									
C13	Common Projects: upsize gravity pipe under Parkside, 8th, Cheney and Oak Island from 8" to 10" or 12"										$352,900	
	Wastewater Treatment											
T1	Aeration Pipe and W. Hunter flowmeter	$21,300										
T2	Headworks Preliminary Engineering	$50,000										
T3	Headworks & Screening		$603,600									
T4	Modify FSL		$736,000									
T5	Biolac Expansion			$2,041,000								
T6	UV System Changes	$391,100										
T7	Process Design					$697,200						
	Water Re-use											
R1	Poplar Tract Assessing	$7,000										
R2	Poplar Harvest and Replant		$36,000	$36,000								
R3	Irrigation Pump Upgrade				$87,700							
R4	U.V. System Changes - Class A						$270,000					
R5	Reuse Distributrion				$50,000			$460,000				

LEGEND:

2008 Value

* All figures include design, construction and contingency costs

City of Veneta

Wastewater Master Plan

CIP Schedule

Table Ex1

WEBER ELLIOTT ENGINEERS, P.C.

Section 3
STUDY AREA CHARACTERISTICS

3.1 STUDY AREA

The city of Veneta is located approximately 13 miles west of the city of Eugene in central Lane County (Figure 3.1.1). The city core is in the north central part of town and parallels Oregon State Highway 126. The main business and residential portion of the city is located about one-quarter mile south of the highway. The Central Oregon and Pacific Railroad runs parallel to and south of Highway 126. Highway 126 is the primary transportation corridor from the Eugene area to the Oregon Coast.

Veneta was incorporated in 1962. The population of the City, as of July 1995, was 2,785 based upon estimates by the Portland State University Center for Population Research and Census. The city experienced considerable variation in population though the 1980's. The estimated low was in 1986 with a population of 2,290. Since 1986 the community has shown a steady increase in population at greater than a 3 percent annual rate. A continued significant growth rate is anticipated because of Veneta's growing appeal as a bedroom community due to the population growth being experienced in the Eugene/Springfield area. The City of Veneta, as of 1990, had 937 housing units with 596 units being owner-occupied.

In addition to serving as a bedroom community, Veneta has a modest commercial and industrial base. Service industries including gas stations, fast food, mini-grocery stores and a shopping center line the Highway 126 corridor and provide commercial services for residents and travelers. For many years the wood products industry provided the primary economic base for Veneta. At one time, International Paper Company operated a saw and veneer mill in Veneta.

The city currently has an art gallery, dollar store, tanning salon, video store, four restaurants, two banks, two convenience stores, a florist, cleaners, deli, antique, and used clothing shops. The most visible retail development is the 103,000-square-foot West Lane Shopping Center north of and along Highway 126. The Fern Ridge School District is also a significant employer. The school district has two schools within the city limits and has discussed the possibility of a third site.

The Facilities Plan study area is contiguous to the urban growth boundary and is shown on Figure 3.1.2.

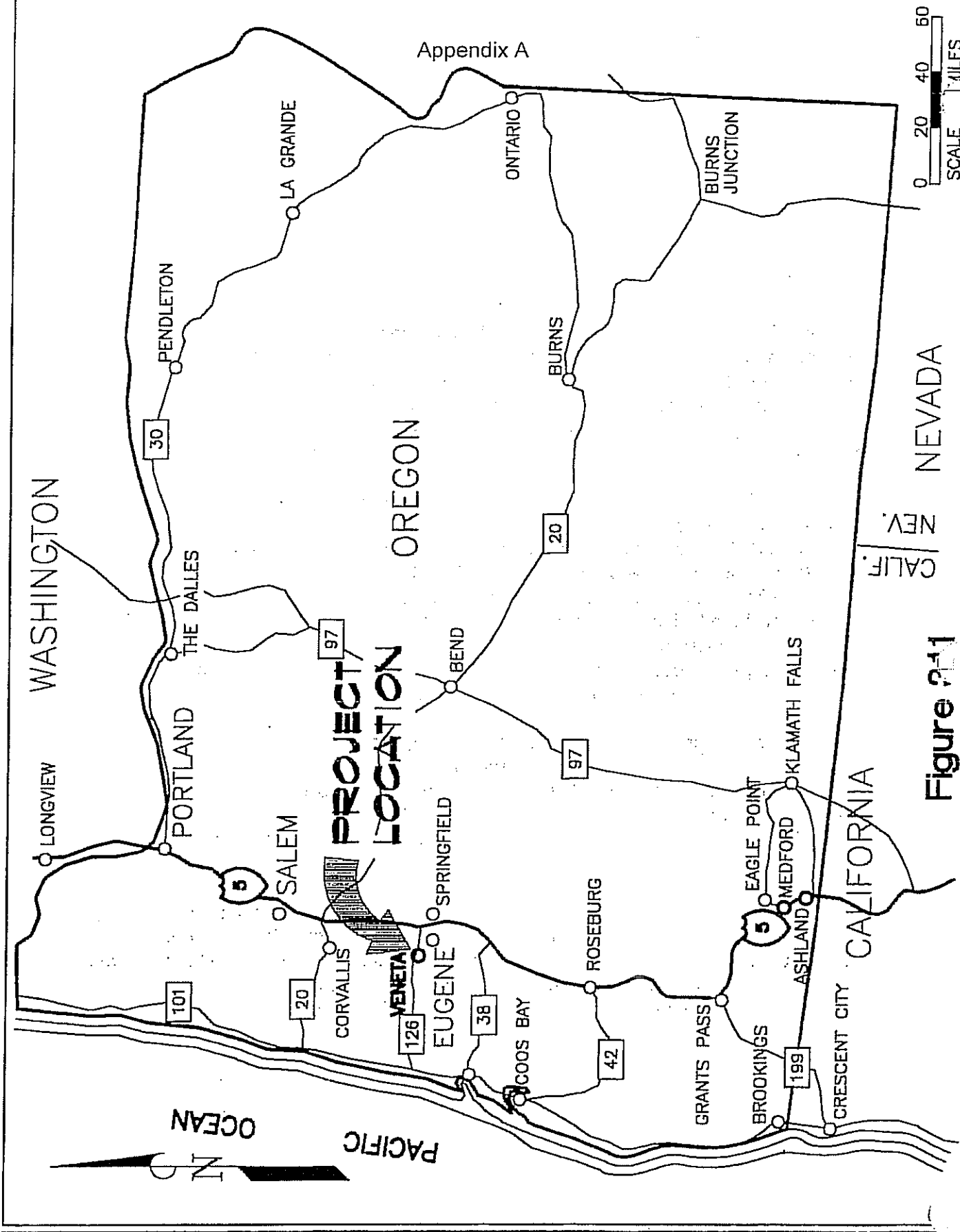


Figure 3-1

S:\S\C\3033.12\LOCATION.dwg - 06/13/96-15:27 - KMF

SYSTEMS WEST ENGINEERS, INC. 1800 Valley River Drive Suite 310 Eugene, Oregon 97401 Phone (541) 342-7210 Fax (541) 342-7220	FIGURE 3.1.1 LOCATION MAP WASTEWATER FACILITIES PLAN LANE COUNTY VENETA, OREGON	Page 3-2 PROJECT 3033.12

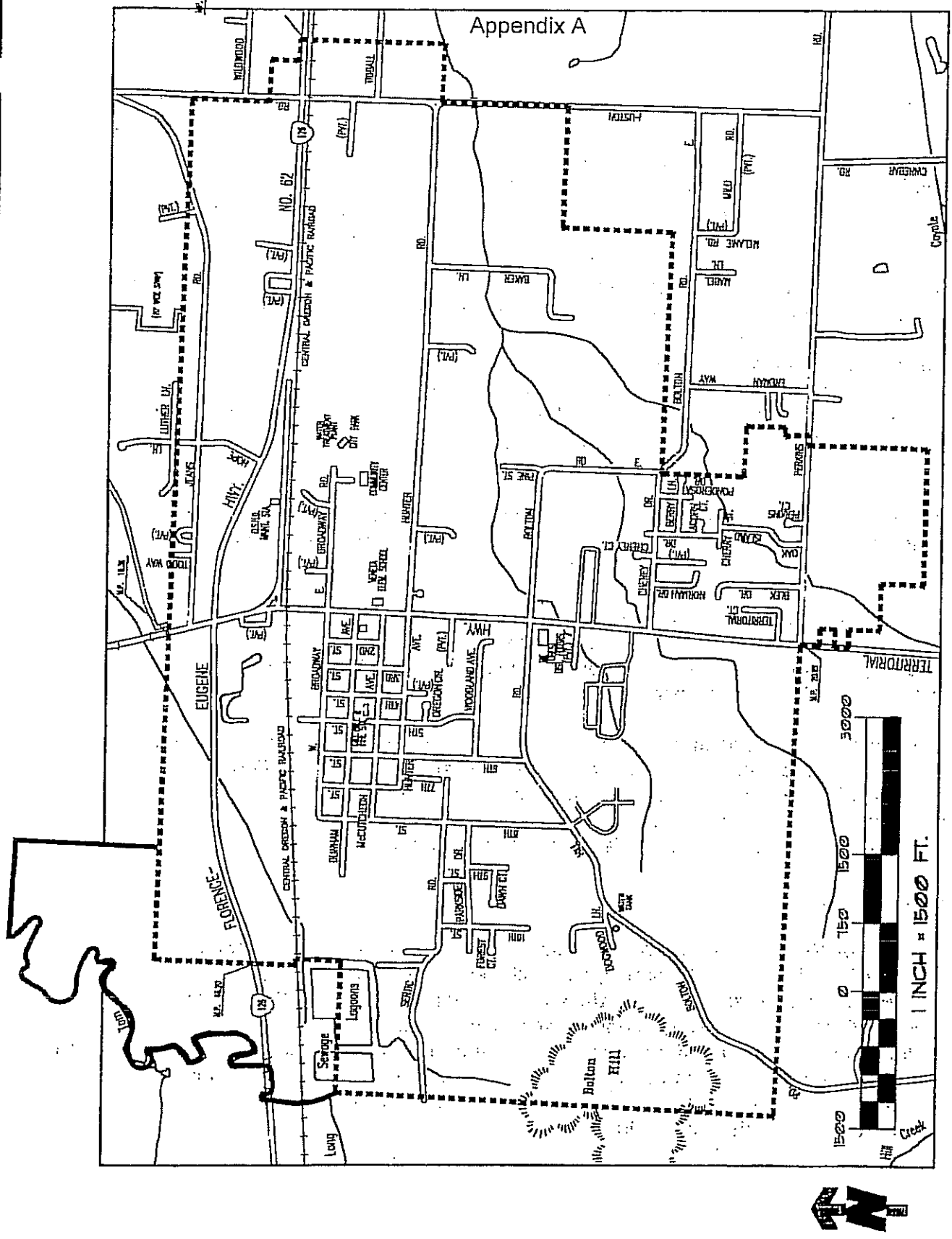


Figure 3.1.2

SYSTEMS WEST ENGINEERS, INC.
1900 Valley River Drive
Suite 310
Eugene, Oregon 97401
Phone (541) 342-7210
Fax (541) 342-7220

FIGURE 3.1.2 STUDY AREA
WASTEWATER FACILITIES PLAN
LANE COUNTY
VENETA, OREGON

PROJECT 3033.12

3.2 PHYSICAL ENVIRONMENT

3.2.1 Climate

Veneta is located in the southwest portion of the Willamette Valley. It has essentially identical climatic conditions to the Eugene/Springfield area. The summer months and into the fall are typically warm, dry, and sunny. Late fall, winter and early spring are characterized by damp, cloudy and cool conditions.

The annual rainfall typically exceeds 45 inches with about 5 inches occurring outside a November through April window. This is primarily due to the strong influence of marine air carried inland by prevailing winds. In most cases, summertime rainfall results from thunderstorm activity carried over from the coastal range or as a precursor to air turbulence caused by the Cascade range to the east. Typically snowfall is light. There was no accumulated snow in the 1993-1994 winter and only one accumulated snow event in the 1994-1995 winter. Historically, Eugene has two days a year with at least one inch of snow on the ground.

Due to the strong influence of marine air, temperatures are moderate. The average winter temperature in nearby Eugene is 42°F with average daily minimum temperatures of 35°F. Summer temperatures average 64°F with an average daily maximum of about 78°F.

Winds are normally light with occasional high winds as a result of localized thunderstorm activity. An occasional storm will develop in the coastal regions and extend inland providing significant gusts. The prevailing winter wind is from the north-northwest at an average speed of 8 miles per hour.

A prevalent winter condition for the Veneta vicinity is fog. Pockets of fog will lie in the lower portions of the community and extend easterly across the Fern Ridge Reservoir. The combination of high humidity and nocturnal cooling associated with winter wet weather traps moist air in the low-lying areas to create the foggy conditions. The fog generally dissipates when sufficient solar radiation heats the moist air.

3.2.2 Soils

The city of Veneta sits on three prevalent landforms with characteristic soil types. The first landform is the river lowlands, generally represented by the area adjacent to the Long Tom River including the area near the wastewater treatment facilities and the wastewater land application area. The predominant soil is McBee silty clay loam. This moderately well drained soil is a flood plain soil. It is characterized by a dark, grayish-brown, silty-clay loam about 24 inches thick. The subsoil is dark grayish-brown and brown, mottled silt loam about 17 inches thick. This soil has moderate permeability. The effective rooting depth is limited by a high water table.

A Linslaw loam and Salkum silty clay loam are the other soils found in the river area. The Linslaw loam is a deep somewhat poorly drained soil found in old terraces and colluvial fans. In addition to being found near the Long Tom River, Linslaw loam is also predominate along small intermittent streams in the eastern and southern portions of the study area. The Linslaw soil has a slow permeability.

The Salkum silty clay loam is found in an isolated higher outwash terrace north of Highway 126. This well drained soil comprises very little of the area in the vicinity of Veneta.

South of the Long Tom River soils are old alluvial and glacial outwash terraces comprised of Salkum silt loam, Veneta loam, and Dupee silt loam. Veneta loam is a deep, moderately well drained soil formed in mixed alluvium. It is the predominant soil type of eastern Veneta as well as the gently sloped side of town.

Salkum silt loam is a deep, well-drained soil generally limited to the commercial district of the downtown area. The Dupee silt loam is generally found on steeper (3 to 20 percent slopes) on the eastern toe of Bolton Hill. This poorly drained soil is underlain by a dark brown silty clay and clay loam. It has moderately low permeability and low strength.

The hillside area, which generally represents the eastern and northern toe of Bolton Hill, contains Bellpine silty clay loam. This soil is prevalent on slopes ranging from 3 to over 25 percent. It is a moderately deep, well-drained soil with low permeability and is underlain by a weathered tuffaceous sandstone.

3.2.3 Geologic Hazards

Known geologic hazards within the study area include excessive slopes, high seasonal groundwater, flooding, and stream erosion.

Excessive Slopes. The steep slopes in Veneta are located on the western and southwestern area of the city adjacent to Bolton Hill. Steep slopes can have the potential for mass movement and slope erosion. Mass movements result from shifting of rock or soil material in response to gravity, such as landslides and rock slides. Evidence of recent mass movement is not apparent in the Veneta area, although a recent geologic report suggests that, on some steep terrain, soil creep is believed to occur.

Slope erosion is the removal of soils or weathered bedrock which occurs as a result of sheet wash, rill erosion or gully erosion. Recent evidence of minor occurrences of slope erosion can be seen in the Bolton Hill area. This is primarily caused by private land use practices (mainly logging and road construction) which can exacerbate slope erosion.

High Groundwater. Seasonal high groundwater is a common occurrence for this area of Lane County. The core community area, western, and eastern portions of the community all contain soils which are prone to exhibiting seasonal high groundwater. The occurrence of high groundwater is caused mostly by perched water tables due to soil saturation and, therefore, represents a short-term condition.

Flooding. Upstream of the Fern Ridge Reservoir, the lowlands along the Long Tom River are subject to periodic overflow and drainage problems when heavy winter rainfall causes the river to flood or causes excess water to build up in or on the soil. Major floods on the Long Tom River occurred on December 22, 1955, December 22, 1964, and February 8, 1996. The 1955 flood had a peak discharge of 6,990 cubic feet per second (cfs), the 1964 flood discharge was 6,450 cfs, and the 1996 peak discharge was 4,830 cfs. The 100 year flood boundary for the Veneta area is shown in the Appendix. The city has an existing ordinance which controls and restricts activities and development within the 100 year floodplain.

Stream Erosion. Moderate levels of bank erosion have been observed within the Long Tom River channel. The stream edge near the city's sewage outfall seems to experience considerable pedestrian traffic due to its close proximity to Highway 126. This high volume of human travel tends to loosen vegetation and soils.

3.2.4 Public Health Hazards

Proper management and operation of a wastewater collection and treatment facility is critical in preventing public health hazards when failures occur. The city has four identified failure points. The first three points are the pump stations. Failure at any of these facilities can lead to raw sewage spills or backup of sewage into homes and businesses. The actual extent of the risk to public health is dependant upon the nature and duration of the failure, the sewage flowrate, and the response time of the city's Public Works Department.

In addition to the public health hazards associated with the pump stations, there is a fourth possible failure point associated with the wastewater treatment system itself. In the winter of 1995-1996 the existing lagoons were within one inch of overflowing. The following winter some overflow occurred. If significant overflow had occurred, a large wetland area with extensive human access would have been flooded with partially treated sewage.

The city has not experienced any significant failures in recent years which presented a public health hazard.

3.2.5 Energy Production and Consumption

The city's Comprehensive Plan does not identify any major energy resources in the Study Area.

Energy consumption is expected to increase within the Study Area due to population growth during the planning period and may increase slightly as a result of any wastewater treatment plant improvements. The Study Area is served by two electrical utilities. Most of the northern portion of the community is served by Lane Electrical Cooperative Inc. and the remainder of the community is served by Emerald People's Utility District.

3.2.6 Water Resources

Both surface and groundwater resources are used in the Study Area.

Surface Water. The Long Tom River is the only surface water source within the Study Area which flows year round. The Long Tom River drains an area of about 100 square miles, generally west and northwest of Veneta. Most of the Long Tom River drainage is forested. The river originates in the coastal mountains and passes through forested area, agricultural lands and small unincorporated communities. The U.S. Army Corps of Engineers has identified the Long Tom River as a major contributor of pollution into Fern Ridge Reservoir.

Fern Ridge Reservoir is an Army Corps of Engineers flood control and recreational facility located northeast of Veneta. The Long Tom River is the only tributary to the Fern Ridge Reservoir which flows year round.

Groundwater. Groundwater serves as the drinking water source for the city of Veneta as well as adjacent rural properties. With the overlying silty and clay soils in conjunction with sandstone bedrock, the aquifer has a limited ability to provide water. Some localized alluvial deposits provide a modest yield. The well water has consistently met or exceeded state drinking water standards but is high in iron and manganese. These metals tend to give the water an unsightly reddish and blackish appearance. The city has abandoned a well which produced water high in total dissolved solids.

In July of 1988, an engineering study for Central Road Water summarized and documented groundwater resource problems in the general area south and east of Veneta. The report explains that many homes in the area do not have an adequate water supply in the summer months, and states that as time goes on "the condition will worsen as the groundwater table lowers" and ... "wells decrease in yield." In 1983, the City of Veneta commissioned an independent engineering consultant to investigate well conditions in the Veneta area. That report substantiates concern about iron in well water.

3.2.7 Flora and Fauna

Flora. The natural vegetation within the study area has been largely replaced by urban and rural residential or agricultural (pasture) uses. The area generally supported conifer forests but to a large extent these have been removed.

Typical native vegetation along the riparian areas and hillsides near Bolton Hill include such tree species as Douglas fir, big leaf maple, California black cottonwood, Pacific yew, ash, and Oregon oak. Shrubs which can be found are snowberry, indian plum and western hazel. Willows and various grasses are also found in this habitat as well as *Lomatium bradshawii*. *Lomatium bradshawii* is a federally listed endangered plant species. It is endemic to the wetland areas of Western Oregon and Veneta.

Fauna. A variety of wildlife species are found throughout the study area. Wildlife in the area includes squirrels, skunks, raccoons, nutrias, coyotes, opossum, deer, and a variety of reptiles and amphibians. An occasional roaming cougar has been seen in rural portions of the urban area. The riparian habitat along the small drainage channels also provides nesting area and cover for baldpates, pintails, teals, scaups, mallards, and other duck species.

Common species of fish in the Long Tom River include the large mouth bass, bluegill, white and black crappie, cutthroat trout, dace, and sculpin. These fish are also found in the nearby Fern Ridge Reservoir. The Fern Ridge Dam serves as a block to passage of coho salmon, chinook salmon, and steelhead. Discussions with long-time residents indicate that the upper reaches of the Long Tom River probably never contained anadromous salmonids.

3.2.8 Air Quality and Noise

Air Quality. The existing air quality in Veneta is good. The Lane Regional Air Pollution Authority's (LRAPA) report cites slash and field burning practices as significant intermediate air pollution sources. LRAPA does not monitor air quality in Veneta but does have authority for the Veneta airshed. During cold periods with stagnant air, residential wood heating does significantly impact local air quality. The closest air quality monitoring station is located in Eugene.

Noise. Sources of noise in Veneta include Highway 126 and the Central Oregon and Pacific Railroad. The state highway serves as a main thoroughfare for residents of the city and the coastal communities as well as tourist travel. Citizens have recently complained about the noise associated with railroad night operations.

3.2.9 Environmentally Sensitive Areas

Riparian Zone. Riparian zones include the riparian zone adjacent to the Long Tom River as well as incidental riparian zones that are a part of the intermittent drainage streams found within the community. The riparian zones are considered environmentally sensitive because of the variety of vegetative and wildlife species which take advantage of this habitat. The riparian zones provide erosion control, drainage management, wildlife cover and shading for surface waters.

Wetlands. Wetlands are one of the most biologically productive components of the environment. Their functions and value include primary production, water quality improvement, fish and wildlife habitat, flood control, point of entry for groundwater discharge, shoreline anchoring, and erosion control.

The most significant wetland area for Veneta includes the tract at the north extent of the city limits and adjacent to the wastewater summer discharge area. This area was studied by the Pacific Basin Shelter Company, Inc. (PBSC) consulting firm. PBSC is evaluating educational and environmental opportunities available for the wetlands in the northern portion of Veneta and issued a summary report entitled "Ecological Planning for the City of Veneta, the Oregon Country Fair, and the Upper Long Tom Watershed", in September, 1996.

Historical and Archaeological Sites. Veneta has no historical sites identified in the comprehensive plan. Archaeological sites have been identified which appear to be associated with the processing of camas. A camas oven has been located just outside the Veneta city limits and was found to be 8,900 years old by carbon dating methods. Other possible indian occupation sites show carbon dated materials which are about 4,000 years old.

Threatened or Endangered Species. A comprehensive inventory for threatened or endangered species in Veneta has not been completed. Currently, *Lomatium bradshawii*, an endangered plant, has been documented as growing in wetland areas north of Veneta. No other threatened or endangered species are known to reside in the Study Area; however, a biological inventory has not been completed. If, as a result of implementing the proposed alternative, new (previously not surveyed) land sites are opened for construction of some part of the collection/treatment system, it will be necessary to perform both historical/archaeological and biological surveys to assure that impacts to threatened or endangered species and historical valued sites do not occur.

3.3 SOCIO-ECONOMIC ENVIRONMENT

The future need for wastewater conveyance and treatment within Veneta will depend, in large part, on socio-economic conditions within the city as well as conditions in neighboring Eugene/Springfield. Veneta's location as a bedroom community for the

Eugene/Springfield area will continue to provide much of its livelihood. In this section, general economic conditions, trends, population, land use, and public facilities relating to central Lane County will be discussed.

3.3.1 Economic Conditions and Trends

Population growth and future wastewater generation in Veneta will likely be affected by regional economic conditions and trends. Major industrial and commercial development has been proceeding in the Eugene/Springfield area for the last half decade, and it is anticipated to continue for the immediate future.

Veneta is an attractive place to live because it reflects a largely rural flavor which sets it apart from the more urban character of the Eugene area. The city also lies within a reasonable commuting distance to an urban area, has basic commercial amenities, affordable housing, and a K-12 school system. Of these attributes, the proximity to the Eugene area and cost of housing will likely stimulate continued growth within the Veneta area.

The City of Veneta has participated in a strategic planning process through Rural Development Initiatives (RDI). The RDI process is intended to produce an economic development plan for the community. This process has identified key issues in the community and has prepared a strategic plan for future economic development. The City of Veneta understands that a high degree of economic dependency exists with the cities of Eugene and Springfield and that economic autonomy is not a realistic goal. The Veneta economy continues to be highly dependent on state and regional economic trends.

Currently, Veneta has no significant industrial or commercial development proposals pending. One potential employer has discussed the possibility of constructing a tile roof manufacturing facility with the City Planning Commission. As many as 12 jobs were proposed. Recent economic development in Eugene has focused on west Eugene industrial development. The Hyundai industrial development would be less than 10 miles from Veneta, may generate over 1,000 jobs and could have a huge impact on the community.

Location. Veneta is only a 20-minute drive from many major commercial and industrial areas of Eugene. It is 30 minutes from new and attractive employment opportunities being developed in Springfield. It is less than an hour from the recreational opportunities of the coast. The City's proximity to these facilities and opportunities is considered desirable by many people. Veneta will continue to be a bedroom community for Eugene/Springfield.

Housing. The cost of housing is generally much more affordable in Veneta than the nearby Eugene/Springfield area. In 1990, a median owner-occupied house in Veneta was valued at $54,300. New homes in Veneta can cost between $125,000 and $450,000 and tend to be owned by middle to upper class families.

Veneta should be in a position to continue to offer plentiful and affordable housing well into the future.

3.3.2 Population

The Portland State University (PSU) Center for Population Research and Census has issued a July 1995 population estimate of 2,785 for Veneta; 2,815 estimated for July 1996; and 2,845 estimated for July 1997. The certified census figure based on the 1990 census is 2,519. The PSU 1995 population estimate projects an annual growth rate of 2 percent.

Historical Population. Since Veneta did not become incorporated until 1962 there is no census data specific to Veneta prior to 1962. In 1962, the City's population was 1,125. The City experienced continued and steady growth throughout the 60's and 70's. In 1980 a peak population of 2,449 occurred. The population declined to a low of 2,290 in 1986 and then climbed to 2,470 by 1989.

Future Population. Projected population growth is the basis for determining future wastewater demand and the associated sizing and capacity of the wastewater system components. To estimate future populations, past trends need to be taken into account. Predicting the growth of Veneta is complicated by adjustments for sporadic growth patterns, an economy subject to flux, and changing development interests. As of spring 1995, a total of 500 housing units were being planned in some form or another for the city. Discussions in early 1996 with developers have suggested as many as 680 housing units. Most of these units were proposed for the southwest portion of the city. By late 1995, only two subdivisions with a total of 40 lots had been submitted for tentative approval by the planning commission. The potential for considerable housing development is large.

Lane County planners and population estimators for the state of Oregon have generally acknowledged a 2 percent annual increase in population as reasonable. The City of Veneta's 1989 comprehensive plan estimated an urban service area population of 5,944 by the year 2010 based on a 3 percent annual growth rate.

Figure 3.3.1. and Table 3.3.1 illustrate the projected growth rate for the city of Veneta which will be used in this Veneta Wastewater Facilities Plan for the period 1997 through 2017. This population growth scenario and others were presented to and discussed by the Veneta City Council on February 12, 1996. The council determined that the selected growth estimate shows the most likely projection of future growth trends. This figure conveys fast initial growth followed by moderately high growth. Recent developments in west Eugene in regard to job and economic opportunity led the council to believe that a high growth rate is more appropriate for Veneta.

Sewered Population. Not all homes within the city limits are connected to the sewer system. The area between Hunter Road and Cheney Road and east of Territorial Highway is not sewered at this time. In addition, areas east along Hunter Road do not have existing sewer service, and homes on Bolton Hill and the area south of Perkins Road are not connected to the city sewer system.

A 1994 approved subdivision on East Hunter Road included 28 lots which contain 0.33 to 0.50 acres each. These lots are scheduled for septic systems. Aerial photographs of Veneta were reviewed to determine the extent of population not served by sewers. The aerial photographs were compared to maps of sewered areas, and house counts were made. It was determined that 100 homes are not connected to the city sewer system. With an average household of 2.89, this means 289 people are not currently served by the sewer system. The actual population receiving service in 1995 was 2,785 less 289 or 2,496.

TABLE 3.3.1 PROJECTED SEWERED POPULATION FOR CITY OF VENETA

Year	Anticipated Growth/Year	Projected Sewered Population
1995	5%	2,496
1997	5%	2,752
2002	3%	3,512
2007	3%	4,071
2012	3%	4,719
2017	3%	5,471

Appendix A

PROJECTED VENETA SEWERED POPULATION GROWTH

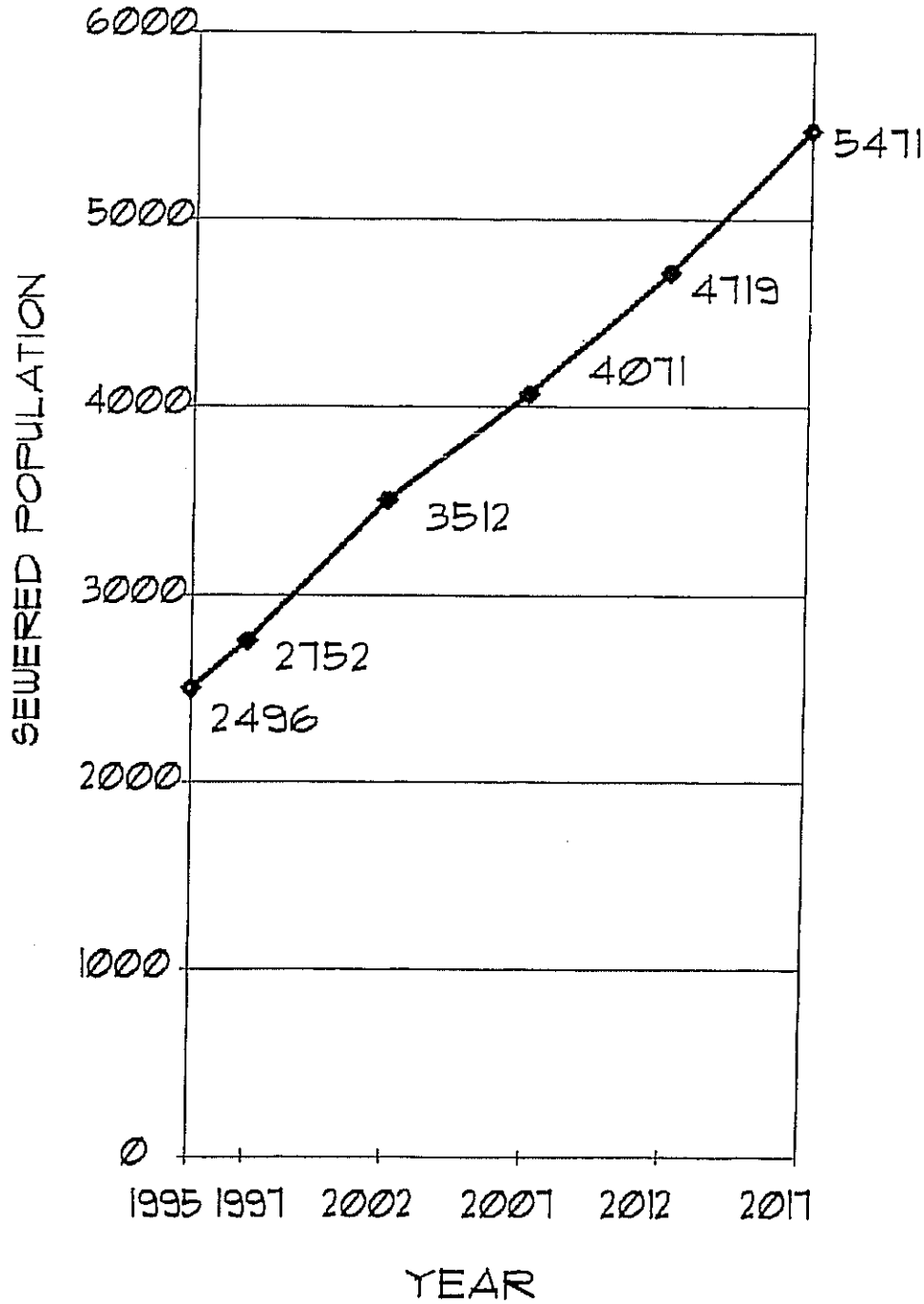


Figure 3.3.1

S:\S\C\3033.12\POPGROTH.dwg - 06/13/96-16:55 - KMF

SYSTEMS WEST ENGINEERS, INC.
1600 Valley River Drive
Suite 310
Eugene, Oregon 97401
Phone (541) 342-7210
Fax (541) 342-7220

FIGURE 3.3.1 VENETA POPULATION GROWTH
WASTEWATER FACILITIES PLAN

LANE COUNTY
VENETA, OREGON

PROJECT 3033.12

Appendix A

APPENDIX B – For use with Chapter 3

B.1 Sewershed Map

This section contains the map referenced by Sections 3.2.5 and 3.4 of Chapter 3.

The map (Figure B1) depicts the locations of the sewersheds and their subsheds, as well as the positions of manholes, sewer pipes and pumps. The map also shows the locations of flooded, or nearly flooded, gravity pipes. "Input branches" represent pipes which acted only as load inputs on the model, and whose physical characteristics were not incorporated into the model. No clean-outs were modeled.

At the effluent manhole for each sewershed and subshed, we inserted graphical plates that indicate the lots served within those sheds. Sewersheds have square plates, and their subsheds have circular plates. The sewershed (square) plates show the total number of lots served within the whole shed. The subshed (circular) plates show the total number of lots served within the subshed, and a running total of lots upstream of that subshed's effluent manhole. For instance, subshed 3 of shed 7 has 93 lots inside of its boundaries, but it is ultimately the effluent manhole of subsheds 4, 5 and 6 too, so the running total of lots served at that node is 383.

The flooded pipes were identified by the modeling software. The software divides the average hydraulic grade line across each pipe by its respective rise. If the model calculates the average HGL over rise to be less than or equal to a certain number, it assigns the pipe a specific color. Table B.1a shows the values used in the model's map:

Table B.1a

Pipe Color Key		
Avg. HGL/Rise	**Color**	**Meaning**
Less than or equal to 75%	Blue	Not flooded
Less than or equal to 100%	Orange	75% covered
Less than or equal to 10,000%	Red	100% (or greater) covered

The intermediate value (75%) was included because the capacity of the sewer may be inadequate in 2030 if the water level in a gravity pipe is calculated to be over 75% full.

The overflowing manholes were located by determining where the hydraulic grade line elevation would rise above the rim elevation of the manhole during peak loading times.

B.2 Data Tables

This section contains tables to help supplement Section 3.2.5 of Chapter 3.

The data in this section represents the physical data acquired from sewer profiles and plan sets of Veneta's sewers. None of this information came from surveys.

Each data table refers to a different element in Veneta's collection network. The elements are:

- Manholes
- Gravity Pipes
- Pressure Pipes
- Pumps
- Wet Wells
- Pressure Junctions
- Outlets

The following section is a field-by-field reference for each data table. Fields not referenced here are considered self-explanatory.

Manholes

Ground/Rim: These are the elevations of the ground at the rim of a manhole, and the elevation of the rim of the manhole, respectively. These were usually considered the same, unless we needed to model a chimney on top of the manhole.

R/G=: This is the logical answer to the question, 'are rim and ground elevations necessarily the same?' This was selected so that, for instance, if one wanted to model a 5 foot chimney (extension) on every manhole, they just needed to change the rim elevations of the manholes to "**=Ground Elevation + 5**," then re-synchronize to the model.

INV IN: This matches the effluent flow line elevation of the pipe upstream of this manhole.

SUMP: This is the elevation of the floor of the manhole. This is somehow different from the flow line elevation of the manhole, perhaps so that one can model a manhole with standing water.

Head loss Method/HEC-22: When specifying that the model should calculate head losses across manholes using HEC-22 methods, one also needs to specify if the benching of the manhole is flat, half-depressed, full-depressed, or a drop. The 'benching' of a manhole is the shape of its floor. Pipes empty their contents onto manhole floors, so the shape of the floor affects sewer flow. Flat benching means the manhole has a flat bottom. Half-depressed means there is a cylindrical depression in the floor to channel water over the bottom of the manhole. Full-depressed means there is a deeper channel in the floor. Drop means the floor of the manhole drops suddenly below the pipe, so water coming out of the pipe splashes onto the floor of the manhole. In Veneta, the manholes are typically half-depressed.

Pattern: This is the multiplier pattern used on the load on the sewer run. For instance, if the base load is 70 GPCPD, and there are 7 lots with 2.89 capita per lot flowing into this manhole, and the pattern multiplier for midnight is 8.6, then at that

hour, that manhole has 70 x 7 x 2.89 x 8.6 = **12178 GPD** of flow. The pattern used by the model is shown in Table B.2a:

Table B.2a

Veneta Peak Day Pattern	
Time from start (hours)	**Pattern Multiplier**
0.00	8.60
1.00	8.72
2.00	8.30
3.00	7.84
4.00	7.40
5.00	7.23
6.00	7.23
7.00	6.86
8.00	6.70
9.00	6.72
10.00	6.88
11.00	6.92
12.00	7.42
13.00	9.68
14.00	9.49
15.00	8.91
16.00	8.65
17.00	8.32
18.00	8.47
19.00	8.71
20.00	8.84
21.00	8.82
22.00	8.69
23.00	8.49

Counted Lots/Lots: The 'Counted Lots' field is the total number of lots counted on the maps prepared by LCOG. The 'Lots' field is the expected number of occupied lots. This is actually determined by the formula "**=Counted Lots*0.9**." The result is rounded up to the nearest whole number.

LOAD (GPD): This is the load in gallons per day on each manhole before a pattern multiplier is applied. The formula is "**=Lots*70*2.89**."

Gravity Pipes

Slope: the slope is calculated for each pipe with the formula "**=100*(Up Inv - Down Inv)/Length**."

Up Inv/Down Inv: the up and down inverts are the influent and effluent flow elevations of the pipe, respectively. The up invert typically corresponds to the

SUMP elevation of the upstream manhole, and the down invert corresponds to the INV IN elevation of the downstream manhole.

Pressure Pipes

PP Inf/Eff: These elevations represent the influent or effluent elevations for pressure pipes in the systems. The force mains do not have entries in the 'Length,' and have influent elevations significantly lower than their effluent elevations.

PP Diam/Mat: This is short-hand for Pressure Pipe Diameter and Material.

Pumps

The pumps in the system are modeled using pump curves. The Pine Street pumps are modeled with a three point pump curve provided by the manufacturer's specifications. The West Lane (or Jean's Road) pumps are modeled using a 'design' or one-point curve, due to the lack of documentation available about the manufacturer of the pumps. The curves for the two pump locations are tabulated in Table B.2b:

Table B.2b

Pine Street Pumps				**West Lane (Jean's Rd.) Pumps**		
	Head (ft)	Discharge (GPM)			Head (ft)	Discharge (GPM)
SHUTOFF	37.00	0		SHUTOFF	69.33	0
DESIGN	25.00	350		DESIGN	52.00	130
MAX	16.00	560		MAX	0.00	260

P RIM: Design head (for middle point of pump curve).

P Inv In/Inv In 2: Effluent elevation of pipes entering wet well (ignored during calculations).

P Inv Out: Ceiling elevation of wet well (ignored during calculations).

P On/Off: Wet well alarm elevations. These entries may be redundant, thus being ignored during calculations.

P Sump: Sump elevation of pump, approximately equal to ground elevation of the pump station.

P Power: Pump power, in horsepower.

Wet Wells

W Ground: Ground elevation of pump station.

W Sump: Sump elevation (floor) of wet well.

W Inf/Inf 2: Effluent elevation of pipes entering wet well.

P On/Off: Wet well alarm levels. During the model simulations, when the wet well level reaches the P On elevation, it should trigger the pump to come on. When the pumps lower the level to the P Off elevation it should turn the pump off. P On

should be higher than P Off, and P Off should be lower than the effluent elevation of the pipes entering the wet well.

Pressure Junctions

PJ Elev: This should be the elevation of the pump.

Outlets

The outlet in the model has a 'tailwater elevation' parameter which was hand-input. This tailwater elevation represents the level of the weir in the treatment plant's wet well, plus a half of a foot due to the sharp-crested weir equation.

ORIM: This is the ground elevation at Veneta's wastewater treatment plant wet well.

O Inv In, Inv Out, Sump: These elements represent both the flowline elevation of the outlet and the sump elevation of the treatment plant wet well.

Job 07-74
Wastewater Master Plan

Manhole Model Data

Weber Elliott Engineers, P.C.

Table B.2c

Name	STA Ah	RIM	Ground	INV IN	SUMP	R=G?	Headloss Method	HEC-22	Pattern	Lots	2030 Lots	LOAD (GPD)
MH 300	1637.5	391.62	391.62	382.23	382.13	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH 301	1274.6	390.43	390.43	382.95	382.85	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH 302	955.7	397.8	397.8	383.59	383.49	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH 303	620.5	392.2	392.2	384.23	384.13	FALSE	HEC-22 Energy	Half	Veneta Peak Day	68	0	13756.4
MH 304	347	396	396	384.7	384.6	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH Meter	150	406.5	406.5	396.7	396.7	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH I-21	5138.75	429.83	429.83	406.66	406.66	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	106	22455.3
MH I-20	4825	422.53	422.53	407.04	407.04	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH I-19	4475	422.63	422.63	407.46	407.46	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	22	5057.5
MH I-18	4140	416.63	416.63	407.85	407.85	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH I-17	4115	412.35	412.35	407.88	407.88	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH I-16	3825	412.73	412.73	408.2	408.2	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	0	1416.1
MH I-15	3583	414.23	414.23	408.71	408.56	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	0	1011.5
MH I-14	3407	421.63	421.63	408.98	408.98	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH I-13	3160	424.13	424.13	409.28	409.28	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	16	4652.9
MH I-12	2899	422.53	422.53	409.59	409.59	FALSE	HEC-22 Energy	Half	Veneta Peak Day	9	0	1820.7
MH I-11	2649	420.83	420.83	409.89	409.89	FALSE	HEC-22 Energy	Half	Veneta Peak Day	4	0	809.2
MH I-10	2437	427.83	427.83	410.37	410.2	FALSE	HEC-22 Energy	Half	Veneta Peak Day	10	0	2023
MH I-9	2057	428.73	428.73	410.95	410.95	FALSE	HEC-22 Energy	Half	Veneta Peak Day	22	0	4450.6
MH I-8	1750	428.83	428.83	411.4	411.4	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	0	1011.5
MH I-7	1458	428.73	428.73	411.86	411.86	FALSE	HEC-22 Energy	Half	Veneta Peak Day	20	0	4046
MH I-6	1175	426.63	426.63	412.27	412.27	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	0	1416.1
MH I-5	838.5	425.83	425.83	413.23	413.23	FALSE	HEC-22 Energy	Half	Veneta Peak Day	12	1	2629.9
MH I-4	901	425.63	425.63	412.68	412.68	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH I-3a	755	425.33	425.33	413.03	413.03	FALSE	HEC-22 Energy	Half	Veneta Peak Day	4	0	809.2
MH I-3	603	423.63	423.63	413.4	413.4	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	0	1011.5
MH I-2	182	424.03	424.03	414.43	414.43	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	2	1416.1
MH I-1	44	418.03	418.03	415.03	415.03	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	1	404.6
MH Ba-1	403.1	422.93	422.93	414.43	414.43	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	2	1820.7
MH Ba-2	750.9	421.58	421.58	415.56	415.56	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH Ba-3	1039.4	421.23	421.23	416.43	416.43	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	0	1011.5
MH Ba-4	1360.8	423.33	423.33	417.71	417.71	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	13	4046
MH Ba-5	1884.5	425.73	425.73	419.87	419.81	FALSE	HEC-22 Energy	Half	Veneta Peak Day	157	0	31761.1
MH Ba-6	341	428.63	428.63	421.2	421.1	FALSE	HEC-22 Energy	Half	Veneta Peak Day	23	35	11733.4
MH E-1	255	422.63	422.63	415.73	415.73	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	0	1416.1
MH E-2	552	425.83	425.83	419	419	FALSE	HEC-22 Energy	Half	Veneta Peak Day	11	0	2225.3
MH E-3	887	431.33	431.33		423.03	FALSE	HEC-22 Energy	Half	Veneta Peak Day	4	0	809.2
MH C-1	232.6	418.73	418.73	416.15	416.15	FALSE	HEC-22 Energy	Half	Veneta Peak Day	6	0	1213.8
MH C-2	525	422.73	422.73	417.33	417.33	FALSE	HEC-22 Energy	Half	Veneta Peak Day	8	0	1618.4
MH C-3	825	425.33	425.33	418.53	418.53	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	3	1618.4

Job 07-74
Wastewater Master Plan

Manhole Model Data

Weber Elliott Engineers, P.C.

Name	STA Ah	RIM	Ground	INV IN	SUMP	R=G?	Headloss Method	HEC-22 Pattern		Lots	2030 Lots	LOAD (GPD)
MH C-4	1160	426.83	426.83	419.81	419.81	FALSE	HEC-22 Energy	Half	Veneta Peak Day	33	0	6675.9
MH C-5	1595	427.83	427.83	421.6	421.6	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	-1	202.3
MH H-1	120	417.83	417.83	411.21	411.21	FALSE	HEC-22 Energy	Half	Veneta Peak Day	4	0	809.2
MH H-2	255	421.06	421.06	411.75	411.75	FALSE	HEC-22 Energy	Half	Veneta Peak Day	10	0	2023
MH H-3	548	423.33	423.33	412.92	412.92	FALSE	HEC-22 Energy	Half	Veneta Peak Day	11	0	2225.3
MH H2-Clean	187	428.03	428.03		421.1	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH H2-1	138	425.93	425.93	415.5	415.5	FALSE	HEC-22 Energy	Half	Veneta Peak Day	24	0	4855.2
MH H2-2	429	430.03	430.03	416.66	416.66	FALSE	HEC-22 Energy	Half	Veneta Peak Day	9	0	1820.7
MH H2-3	706	429.17	429.17	417.77	417.77	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH H2-4	876.8	423.83	423.83	418.46	418.46	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	1	1618.4
MH H2-5	1240	433.13	433.13	423.54	423.54	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH H2-6	1403	431.33	431.33		424.19	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH H2-7	390	429.43	429.43	420.76	420.76	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH H2-8	545	430.43	430.43	421.53	421.53	FALSE	HEC-22 Energy	Half	Veneta Peak Day	6	0	1213.8
MH H2-9	678	431.83	431.83		422.06	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH H3-Clean	145	426.94	426.94		418.72	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH H2-3-Clean	150	433.33	433.33		425.33	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH H2-8-Clean	225	429.03	429.03		422.75	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH F-1	135	428.93	428.93	417.33	417.33	FALSE	HEC-22 Energy	Half	Veneta Peak Day	10	0	2023
MH F-2	426	427.73	427.73	418.5	418.5	FALSE	HEC-22 Energy	Half	Veneta Peak Day	9	0	1820.7
MH F-3	315	431.53	431.53		423.54	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH F-4	329	429.33	429.33	422.28	422.28	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	0	1011.5
MH F2-Clean	161	431.03	431.03		424.94	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH 244	384.11	424.73	424.73	413.55	413.45	FALSE	HEC-22 Energy	Half	Veneta Peak Day	10	0	2023
MH G-1	349	427.33	427.33		421.33	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH B-1	231.1	421.43	421.43		415.95	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	2	404.6
MH H25-A	38.6	432.33	432.33	423.73	423.73	FALSE	HEC-22 Energy	Half	Veneta Peak Day	9	0	1820.7
MH H25-B	230.5	433.52	433.52	427.93	427.93	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH H22-A	96.5	431.19	431.19	417.51	417.39	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	0	1011.5
MH H22-B	269.1	426.32	426.32	420.82	420.82	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH JR-1	45	387.05	387.05	375.21	375.13	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH JR-2	267.17	385.9	385.9	376.17	376.17	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH JR-3	541.35	387.04	387.04	377.34	377.32	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH JR-4	895	386.4	386.4	378.84	378.83	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH JR-5	1030	386.63	386.63		379.37	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH A-1	0	407.83	407.83		401.83	FALSE	HEC-22 Energy	Half	Veneta Peak Day	35	0	7080.5
MH A-2	187	412.03	412.03	401.1	401.1	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH A-3	436	404.63	404.63	397.29	397.29	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH A-4	634	396.13	396.13	384.92	384.92	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH A-5	970	387.73	387.73	376.53	376.33	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH A-6	1265	385.73	385.73	377.71	377.71	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3

Job 07-74
Wastewater Master Plan

Manhole Model Data

Weber Elliott Engineers, P.C.

Name	STA Ah	RIM	Ground	INV IN	SUMP	R=G?	Headloss Method	HEC-22 Pattern		Lots	2030 Lots	LOAD (GPD)
MH A-7	1485	386.33	386.33	378.59	378.59	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH A-8	170	384.53	384.53	375.05	375.05	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH H3-1	542	425.33	425.33	414.6	414.6	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH H3-2	174.3	421.38	421.38	416.02	416.02	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH H3-3	732	422.83	422.83	416.52	416.52	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH T-1	114689.5	428.23	428.23	421.42	421.39	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	0	1011.5
MH T-2	115038	427.1	427.1	422.87	422.87	FALSE	HEC-22 Energy	Half	Veneta Peak Day	4	132	27512.8
MH T-3	115450	429.55	429.55	424.51	424.51	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	3	2023
MH T-4	115850	433.23	433.23	426.11	426.11	FALSE	HEC-22 Energy	Half	Veneta Peak Day	9	49	11733.4
MH T-5	116250	433.83	433.83	427.71	427.71	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	0	1011.5
MH T-6	116600	435.86	435.86		429.11	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	1	606.9
MH Ch-1	235	428.43	428.43		421.53	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH Ch-2	671	423.37	423.37	414.52	413.07	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	0	1011.5
MH Ah-2	561	425.5	425.5	415.11	415.11	FALSE	HEC-22 Energy	Half	Veneta Peak Day	15	0	3034.5
MH Ah-3	487.5	426.67	426.67	416.52	416.49	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	0	1416.1
MH Ah-4a	860	428.37	428.37	418.08	418.05	FALSE	HEC-22 Energy	Half	Veneta Peak Day	14	0	2832.2
MH Ah-4	983	430.04	430.04	418.43	418.31	FALSE	HEC-22 Energy	Half	Veneta Peak Day	8	0	1618.4
MH Ah-5	1366	432.19	432.19	420.08	420.04	FALSE	HEC-22 Energy	Half	Veneta Peak Day	8	0	1618.4
MH Ah-6	1786	430.92	430.92	421.75	421.71	FALSE	HEC-22 Energy	Half	Veneta Peak Day	6	3	1820.7
MH Ah-7	2073	433.74	433.74	422.87	422.86	FALSE	HEC-22 Energy	Half	Veneta Peak Day	26	0	5259.8
MH Bh-1	0	427.43	427.43	419.36	419.33	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	0	1416.1
MH Bh-2	258.5	425.61	425.61	420.48	420.48	FALSE	HEC-22 Energy	Half	Veneta Peak Day	8	0	1618.4
MH Bh-3	619.5	428.36	428.36	421.89	421.89	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	0	1416.1
MH Bh-4	701.5	428.55	428.55	422.33	422.33	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH Bh-5	860.5	430.83	430.83	423	422.87	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH 501	453	423.01	423.01	412.53	412.43	FALSE	HEC-22 Energy	Half	Veneta Peak Day	9	0	1820.7
MH 502	860	421.69	421.69	411.23	411.13	FALSE	HEC-22 Energy	Half	Veneta Peak Day	11	19	6069
MH D-1	282	425.43	425.43	417.32	417.31	FALSE	HEC-22 Energy	Half	Veneta Peak Day	6	0	1213.8
MH D-2	486	428.25	428.25	418.18	418.14	FALSE	HEC-22 Energy	Half	Veneta Peak Day	9	0	1820.7
MH D-3	688	428.09	428.09	418.96	418.96	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	23	4652.9
MH D-4	191	427.26	427.26	419.16	419.15	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH Bh1-Clean	138.5	424.83	424.83		419.85	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH 500	15.58	423.7	423.7	413.01	412.91	FALSE	HEC-22 Energy	Half	Veneta Peak Day	4	0	809.2
MH 503	816.78	420.02	420.02	409.5	409.4	FALSE	HEC-22 Energy	Half	Veneta Peak Day	26	2	5664.4
MH 234	1106.72	423.75	423.75	408.59	408.49	FALSE	HEC-22 Energy	Half	Veneta Peak Day	6	0	1213.8
MH 235	1428.9	417.38	417.38	407.63	407.53	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	1	1213.8
MH 236	1750.9	417.31	417.31	406.82	406.72	FALSE	HEC-22 Energy	Half	Veneta Peak Day	27	39	13351.8
MH 245	768.11	426.59	426.59	414.81	414.71	FALSE	HEC-22 Energy	Half	Veneta Peak Day	6	3	1820.7
MH 246	1152.1	422.97	422.97	416.06	415.96	FALSE	HEC-22 Energy	Half	Veneta Peak Day	13	7	4046
MH 247	1536.1	425.2	425.2	417.31	417.21	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH 248	311.75	422.67	422.67	405.77	405.66	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	0	1416.1

Job 07-74
Wastewater Master Plan

Manhole Model Data

Weber Elliott Engineers, P.C.

Name	STA Ah	RIM	Ground	INV IN	SUMP	R=G?	Headloss Method	HEC-22	Pattern	Lots	2030 Lots	LOAD (GPD)
MH 249	600.85	418.26	418.26	404.52	404.42	FALSE	HEC-22 Energy	Half	Veneta Peak Day	13	0	2629.9
MH 600	22.16	425.25	425.25	407.02	406.92	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	4	1011.5
MH Ba-F	897.73	421.42	421.42	416	416	FALSE	HEC-22 Energy	Half	Veneta Peak Day	27	22	9912.7
MH P-1	133.86	433.07	433.07	424.26	424.16	FALSE	HEC-22 Energy	Half	Veneta Peak Day	10	0	2023
MH P-2	649.61	432.13	432.13		425.91	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH O-1	359.69	435.44	435.44	425.41	425.28	FALSE	HEC-22 Energy	Half	Veneta Peak Day	13	0	2629.9
MH O-2	598.95	435.32	435.32	426.46	426.36	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	0	1416.1
MH O-3	877.88	435.5	435.5	427.72	427.62	FALSE	HEC-22 Energy	Half	Veneta Peak Day	14	0	2832.2
MH O-4	350	437.68	437.68	428.86	428.76	FALSE	HEC-22 Energy	Half	Veneta Peak Day	21	0	4248.3
MH P-3	21	428.11	428.11	421.81	421.81	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH P-4	195.1	431.68	431.68	422.25	422.2	FALSE	HEC-22 Energy	Half	Veneta Peak Day	9	0	1820.7
MH P-5	320.46	432.79	432.79	422.68	422.63	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH P-6	560.69	433.08	433.08	423.56	423.4	FALSE	HEC-22 Energy	Half	Veneta Peak Day	4	32	7282.8
MH P-7	1594.25	431.15	431.15	424.73	424.63	FALSE	HEC-22 Energy	Half	Veneta Peak Day	114	30	29131.2
MH 237	2019.44	417.52	417.52	406.13	406.03	FALSE	HEC-22 Energy	Half	Veneta Peak Day	10	0	2023
MH 238	2410.9	416.67	416.67	404.85	404.75	FALSE	HEC-22 Energy	Half	Veneta Peak Day	9	60	13958.7
MH 239	2813.49	412.25	412.25	403.54	403.44	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH 240	2879.99	412.51	412.51	403.24	403.14	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH Tr-1	1176	415	415	405.63	405.56	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	1	809.2
MH Tr-2	1425.29	415.1	415.1	406.63	406.59	FALSE	HEC-22 Energy	Half	Veneta Peak Day	84	53	27715.1
MH O-5	360	436.58	436.58	430	429.9	FALSE	HEC-22 Energy	Half	Veneta Peak Day	15	0	3034.5
MH 601	376.58	428.6	428.6	408.23	408.18	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	15	3641.4
MH 602	752.66	426.5	426.5	409.41	409.36	FALSE	HEC-22 Energy	Half	Veneta Peak Day	4	65	13958.7
MH 603	1044.66	422.1	422.1	410.42	410.29	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH 604	1277.54	418.9	418.9	411.4	411.35	FALSE	HEC-22 Energy	Half	Veneta Peak Day	34	364	80515.4
MH 10-A	150	421.31	421.31	407.22	407.22	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH 10-B	216.67	430.43	430.43	424.53	424.43	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	0	1416.1
MH 10-C	665.49	472.71	472.71	466.47	466.23	FALSE	HEC-22 Energy	Half	Veneta Peak Day	6	0	1213.8
MH 10-D	695.49	474.7	474.7	468.53	467.13	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	130	26501.3
MH 6-A	400	428.58	428.58	415.43	415.43	FALSE	HEC-22 Energy	Half	Veneta Peak Day	12	0	2427.6
MH 6-B	902	428.63	428.63	417.43	417.43	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	7	2427.6
MH 6-C	1238	429.03	429.03	418.76	418.76	FALSE	HEC-22 Energy	Half	Veneta Peak Day	26	0	5259.8
MH 6-D	1250	430.83	430.83	418.83	418.83	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH 8-A	262	421.16	421.16	414.43	414.43	FALSE	HEC-22 Energy	Half	Veneta Peak Day	68	13	16386.3
MH 8-B	85	422.94	422.94	416.28	416.28	FALSE	HEC-22 Energy	Half	Veneta Peak Day	11	1	2427.6
MH 8-C	500	431.54	431.54	422.33	422.33	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	4	2225.3
MH 8-D	870.6	436.69	436.69	427.98	427.98	FALSE	HEC-22 Energy	Half	Veneta Peak Day	4	3	1416.1
MH 8-E	1198.7	445.97	445.97	433.09	433.02	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	22	4652.9
MH 6-E	296	430.01	430.01	420.19	420.09	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH 8-F	35	448.12	448.12	433.71	433.57	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH 8-G	145	445.14	445.14	434.17	434.01	FALSE	HEC-22 Energy	Half	Veneta Peak Day	12	0	2427.6

Job 07-74
Wastewater Master Plan

Manhole Model Data

Weber Elliott Engineers, P.C.

Name	STA Ah	RIM	Ground	INV IN	SUMP	R=G?	Headloss Method	HEC-22 Pattern		Lots	2030 Lots	LOAD (GPD)
MH 8-H	390	439.12	439.12	434.96	434.86	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH 8-I	525	439.21	439.21	435.25	435.22	FALSE	HEC-22 Energy	Half	Veneta Peak Day	8	309	64129.1
MH 6-F	387	433.51	433.51	421.84	421.74	FALSE	HEC-22 Energy	Half	Veneta Peak Day	15	0	3034.5
MH 6-G	733	433.34	433.34	422.22	422.19	FALSE	HEC-22 Energy	Half	Veneta Peak Day	3	0	606.9
MH 6-H	909	433.07	433.07	423.57	423.12	FALSE	HEC-22 Energy	Half	Veneta Peak Day	12	0	2427.6
MH 6-I	1008.5	434.18	434.18	424.07	423.97	FALSE	HEC-22 Energy	Half	Veneta Peak Day	2	0	404.6
MH 6-J	1119.52	435.28	435.28	424.65	424.57	FALSE	HEC-22 Energy	Half	Veneta Peak Day	12	162	35200.2
MH U-1	671.2	413.51	413.51	404.13	404.03	FALSE	HEC-22 Energy	Half	Veneta Peak Day	43	0	8698.9
MH U-2	117.2	413.5	413.5	404.8	404.7	FALSE	HEC-22 Energy	Half	Veneta Peak Day	6	0	1213.8
MH U-3	142.2	413.69	413.69	405.66	405.4	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	1	202.3
MH U-4	301.4	413.29	413.29	406.66	406.66	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	1	404.6
MH U-5	542.3	414.68	414.68	407.94	407.94	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	2	1416.1
MH U-6	943	416.47	416.47	410.06	410.06	FALSE	HEC-22 Energy	Half	Veneta Peak Day	22	0	4450.6
MH A-9	430	390.43	390.43	380.31	380.31	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH A-10	869.6	399.13	399.13	387.65	387.65	FALSE	HEC-22 Energy	Half	Veneta Peak Day	1	0	202.3
MH A-11	1240.9	404.93	404.93	393.83	393.83	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH A-12	1365.9	402.73	402.73	394.83	394.83	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	102	20634.6
MH A-13	1858.95	426.23	426.23	404.59	404.59	FALSE	HEC-22 Energy	Half	Veneta Peak Day	0	0	0
MH A-14	2419.1	412.83	412.83	406.83	406.83	FALSE	HEC-22 Energy	Half	Veneta Peak Day	5	131	27512.8
MH T-C	307	433.58	433.58	425.75	425.75	FALSE	HEC-22 Energy	Half	Veneta Peak Day	7	1	1618.4
MH T-D	217.7	421	421		418.55	FALSE	HEC-22 Energy	Half	Veneta Peak Day	11	0	2225.3

Table B.2d

Job 07-74
Wastewater Master Plan

Gravity Pipe Model Data

Weber Elliott Engineers, P.C.

Pipe Name	Length	Slope (%)	Diameter	Material	Up Inv	Down Inv
IPS-300	85	0.1529	27 Inch	PVC	382.13	382
300-301	362.9	0.1708	27 Inch	PVC	382.85	382.23
301-302	318.9	0.1693	27 Inch	PVC	383.49	382.95
302-303	335.2	0.1611	27 Inch	PVC	384.13	383.59
303-304	273.5	0.1353	27 Inch	PVC	384.6	384.23
304-Meter	197	6.0914	21 Inch	PVC	396.7	384.7
Meter-I21	177	5.6271	18 Inch	Concrete	406.66	396.7
I21-I20	313.75	0.1211	21 Inch	Concrete	407.04	406.66
10A-I20	150	0.1200	21 Inch	Concrete	407.22	407.04
I19-I18	335	0.1164	21 Inch	Concrete	407.85	407.46
I18-I17	29	0.1034	21 Inch	Concrete	407.88	407.85
I17-I16	290	0.1103	21 Inch	Concrete	408.2	407.88
I16-I15	242	0.1488	21 Inch	Concrete	408.56	408.2
I15-I14	241	0.1120	18 Inch	Concrete	408.98	408.71
I14-I13	247	0.1215	18 Inch	Concrete	409.28	408.98
I13-I12	261	0.1188	18 Inch	Concrete	409.59	409.28
I12-I11	250	0.1200	18 Inch	Concrete	409.89	409.59
I11-I10	233	0.1330	18 Inch	Concrete	410.2	409.89
I10-I9	380	0.1526	15 Inch	Concrete	410.95	410.37
I9-I8	307	0.1466	15 Inch	Concrete	411.4	410.95
I8-I7	292	0.1575	15 Inch	Concrete	411.86	411.4
I7-I6	283	0.1449	15 Inch	Concrete	412.27	411.86
I6-I4	274	0.1496	15 Inch	Concrete	412.68	412.27
I5-I4	62.5	0.1120	8 Inch	Concrete	413.23	413.16
244-I5	384.11	0.1093	21 Inch	PVC	413.45	413.03
I4-I3a	146	0.2397	12 Inch	Concrete	413.03	412.68
G1-I3a	349	2.3782	8 Inch	Concrete	421.33	413.03
I3a-I3	152	0.2434	12 Inch	Concrete	413.4	413.03
I3-I2	421	0.2447	12 Inch	Concrete	414.43	413.4
I2-I1	138	0.4348	12 Inch	Concrete	415.03	414.43
B1-I1	231.1	0.3981	8 Inch	Concrete	415.95	415.03
I5-Ba1	403.1	0.2481	8 Inch	Concrete	414.43	413.43
Ba1-Ba2	347.8	0.3249	8 Inch	Concrete	415.56	414.43
Ba2-BaF	146.83	0.2997	8 Inch	Concrete	416	415.56
Ba3-Ba4	321.4	0.3983	8 Inch	Concrete	417.71	416.43
Ba4-Ba5	523.7	0.4010	8 Inch	Concrete	419.81	417.71
Ba6-Ba5	341	0.3607	8 Inch	Concrete	421.1	419.87
I2-E1	255	0.3922	8 Inch	Concrete	415.73	414.73
E1-E2	297	1.1010	8 Inch	Concrete	419	415.73
E2-E3	335	1.2030	8 Inch	Concrete	423.03	419
I1-C1	232.6	0.3955	8 Inch	Concrete	416.15	415.23
C1-C2	292.4	0.4036	8 Inch	Concrete	417.33	416.15
C2-C3	300	0.4000	8 Inch	Concrete	418.53	417.33
C3-C4	335	0.3821	8 Inch	Concrete	419.81	418.53
C4-C5	435	0.4115	8 Inch	Concrete	421.6	419.81
I11-H1	120	0.4000	8 Inch	Concrete	411.21	410.73
H1-H2	135	0.4000	8 Inch	Concrete	411.75	411.21
H2-H3	293	0.3993	8 Inch	Concrete	412.92	411.75
H2-Clean	187	5.0000	6 Inch	Concrete	421.1	411.75
H3-Clean	145	4.0000	6 Inch	Concrete	418.72	412.92
I10-H21	138	3.0000	8 Inch	Concrete	415.5	411.36
H21-H22	291	0.3986	8 Inch	Concrete	416.66	415.5
H22-H22A	180.5	0.4044	8 Inch	Concrete	417.39	416.66
H23-Clean	150	5.0000	6 Inch	Concrete	425.33	417.83
H23-H24	170.8	0.4040	8 Inch	Concrete	418.46	417.77

Job 07-74
Wastewater Master Plan

Gravity Pipe Model Data

Weber Elliott Engineers, P.C.

Pipe Name	Length	Slope (%)	Diameter	Material	Up Inv	Down Inv
H24-H25	363.2	1.3987	8 Inch	Concrete	423.54	418.46
H25-H25A	38.6	0.4922	8 Inch	Concrete	423.73	423.54
H21-H27	390	1.3154	8 Inch	Concrete	420.76	415.63
H27-H28	155	0.4968	8 Inch	Concrete	421.53	420.76
H28-Clean	225	0.4978	8 Inch	Concrete	422.75	421.63
H28-H29	133	0.3985	8 Inch	Concrete	422.06	421.53
I7-F1	135	3.3333	8 Inch	Concrete	417.33	412.83
F1-F2	291	0.4021	8 inch	Concrete	418.5	417.33
F2-Clean	161	4.0000	6 Inch	Concrete	424.94	418.5
F1-F4	329	1.2006	8 Inch	Concrete	422.28	418.33
F4-F3	315	0.4000	8 Inch	Concrete	423.54	422.28
H25A-H26	124.4	0.3698	8 Inch	Concrete	424.19	423.73
H25A-H25B	230.5	1.6052	8 Inch	Concrete	427.93	424.23
H22A-H23	96.5	0.2694	8 Inch	Concrete	417.77	417.51
H22A-H22B	269.1	1.2300	8 Inch	Concrete	420.82	417.51
Jean-JR1	21	1.0000	8 Inch	PVC	375.13	374.92
JR1-JR2	222.17	0.4321	8 Inch	PVC	376.17	375.21
JR2-JR3	274.18	0.4194	8 Inch	PVC	377.32	376.17
JR3-JR4	353.65	0.4213	8 Inch	PVC	378.83	377.34
JR4-JR5	135	0.3926	8 Inch	PVC	379.37	378.84
Jean-A8	18	0.7222	8 Inch	Concrete	375.05	374.92
A8-A5	170	0.7529	8 Inch	Concrete	376.33	375.05
A6-A7	220	0.4000	8 Inch	Concrete	378.59	377.71
A5-A6	295	0.4000	8 Inch	Concrete	377.71	376.53
A4-A5	336	2.4970	8 Inch	Concrete	384.92	376.53
A3-A4	198	6.2475	8 Inch	Concrete	397.29	384.92
A2-A3	249	1.5301	8 Inch	Concrete	401.1	397.29
A1-A2	187	0.3904	8 Inch	Concrete	401.83	401.1
H3-H31	429	0.3916	8 Inch	Concrete	414.6	412.92
H31-H32	367.7	0.3862	8 Inch	Concrete	416.02	414.6
H31-H33	190	0.8947	8 Inch	Concrete	416.52	414.82
T2-T1	348.5	0.4161	8 Inch	Concrete	422.87	421.42
T3-T2	412	0.3981	8 Inch	Concrete	424.51	422.87
T4-TC	89	0.4045	8 Inch	Concrete	426.11	425.75
T5-T4	400	0.4000	8 Inch	Concrete	427.71	426.11
T6-T5	350	0.4000	8 Inch	Concrete	429.11	427.71
Ch1-Ah2	326	1.9663	8 Inch	Concrete	421.53	415.12
Ah2-Ch2	110	0.5364	8 Inch	Concrete	415.11	414.52
Ah3-Ah2	349.5	0.3948	8 Inch	Concrete	416.49	415.11
Ah4a-Ah3	372.5	0.4107	8 Inch	Concrete	418.05	416.52
Ah4-Ah4a	123	0.1870	8 Inch	Concrete	418.31	418.08
Ah5-Ah4	383	0.4204	8 Inch	Concrete	420.04	418.43
Ah6-Ah5	420	0.3881	8 Inch	Concrete	421.71	420.08
Ah7-Ah6	287	0.3868	8 Inch	Concrete	422.86	421.75
Bh2-Bh1	258.5	0.3985	8 Inch	Concrete	420.48	419.45
P3-Bh2	340	0.3912	8 Inch	PVC	421.81	420.48
Bh4-Bh3	82	0.5122	8 Inch	Concrete	422.33	421.91
Bh5-Bh4	159	0.3396	8 Inch	Concrete	422.87	422.33
500-501	398	0.0955	8 Inch	PVC	412.91	412.53
501-502	407	0.2948	8 Inch	PVC	412.43	411.23
D1-502	282	2.1560	8 Inch	Concrete	417.31	411.23
D2-D1	204	0.4020	8 Inch	Concrete	418.14	417.32
D3-D2	202	0.3861	8 Inch	Concrete	418.96	418.18
D4-D2	191	0.4712	8 Inch	Concrete	419.15	418.25
Bh1-D3	81	0.3580	8 Inch	Concrete	419.33	419.04

Job 07-74
Wastewater Master Plan

Gravity Pipe Model Data

Weber Elliott Engineers, P.C.

Pipe Name	Length	Slope (%)	Diameter	Material	Up Inv	Down Inv
Bh1-Clean	138.5	0.3538	8 Inch	Concrete	419.85	419.36
Ch2-500	15.58	0.3851	8 Inch	PVC	413.07	413.01
502-503	406.47	0.4010	8 Inch	PVC	411.13	409.5
503-234	289.94	0.2794	10 Inch	PVC	409.4	408.59
234-235	322.18	0.2669	10 Inch	PVC	408.49	407.63
235-236	322	0.2205	12 Inch	PVC	407.53	406.82
245-244	384	0.3021	21 Inch	PVC	414.71	413.55
246-245	383.99	0.2995	21 Inch	PVC	415.96	414.81
247-246	384	0.2995	21 Inch	PVC	417.21	416.06
600-248	289.59	0.3971	10 Inch	PVC	406.92	405.77
248-249	289.1	0.3943	10 Inch	PVC	405.66	404.52
U1-Pine	206.15	0.3784	10 Inch	PVC	404.03	403.25
BaF-Ba3	141.67	0.3035	8 Inch	Concrete	416.43	416
T1-Ba6	60	0.3167	8 Inch	Concrete	421.39	421.2
P1-Bh5	133.86	0.8666	8 Inch	PVC	424.16	423
P2-P1	350.39	0.4709	8 Inch	PVC	425.91	424.26
O1-P1	259.69	0.3928	8 Inch	PVC	425.28	424.26
O2-O1	250	0.3800	8 Inch	PVC	426.36	425.41
O3-O2	289.66	0.4005	8 Inch	PVC	427.62	426.46
O4-O3	250	0.4160	8 Inch	PVC	428.76	427.72
Bh3-P3	21	0.3810	8 Inch	PVC	421.89	421.81
P4-P3	95.1	0.4101	8 Inch	PVC	422.2	421.81
P5-P4	125.36	0.3031	8 Inch	PVC	422.63	422.25
P6-P5	240.23	0.2997	8 Inch	PVC	423.4	422.68
P7-P6	355.03	0.3014	8 Inch	PVC	424.63	423.56
236-237	268.54	0.2197	12 Inch	PVC	406.72	406.13
237-238	391.46	0.3014	12 Inch	PVC	406.03	404.85
238-239	402.59	0.3006	15 Inch	PVC	404.75	403.54
239-240	66.5	0.3008	15 Inch	PVC	403.44	403.24
240-Pine	14.19	0.2819	15 Inch	PVC	403.14	403.1
Tr1-238	173.37	0.4095	8 Inch	PVC	405.56	404.85
Tr2-Tr1	249.29	0.3851	8 Inch	PVC	406.59	405.63
O5-O4	260	0.4000	8 Inch	PVC	429.9	428.86
601-600	376.58	0.3080	10 Inch	PVC	408.18	407.02
602-601	376.08	0.3005	10 Inch	PVC	409.36	408.23
603-602	292	0.3014	10 Inch	PVC	410.29	409.41
604-603	232.88	0.3993	8 Inch	PVC	411.35	410.42
I19-10A	200	0.1200	21 Inch	Concrete	407.46	407.22
10B-10A	216.67	5.3076	8 Inch	Concrete	424.43	412.93
10C-10B	448.82	9.2910	8 Inch	Concrete	466.23	424.53
10D-10C	30	2.2000	8 Inch	Concrete	467.13	466.47
6A-I10	400	0.8000	8 Inch	Concrete	415.43	412.23
6B-6A	502	0.3984	8 Inch	Concrete	417.43	415.43
6C-6B	330	0.4030	8 Inch	Concrete	418.76	417.43
6D-6C	18	0.3889	8 Inch	Concrete	418.83	418.76
8A-I16	262	2.3779	8 Inch	Concrete	414.43	408.2
8B-8A	262	0.7061	8 Inch	Concrete	416.28	414.43
8C-8B	415	1.4578	8 Inch	Concrete	422.33	416.28
8D-8C	370.6	1.5246	8 Inch	Concrete	427.98	422.33
8E-8D	328.1	1.5361	8 Inch	Concrete	433.02	427.98
6E-6D	296	0.4257	8 Inch	Concrete	420.09	418.83
8F-8E	89	0.5393	8 Inch	Concrete	433.57	433.09
8G-8F	110	0.2727	8 Inch	Concrete	434.01	433.71
8H-8G	245	0.2816	8 Inch	Concrete	434.86	434.17
8I-8H	135	0.1926	8 Inch	Concrete	435.22	434.96

Job 07-74
Wastewater Master Plan

Other Element Physical Data

Appendix to Chapter 3

Table B.2e

Pressure Pipes

PP Name	PP Length	PP Inf	PP Eff	PP Diam	PP Mat
Jean Pressure		374.92	415.23	6	Asbestos Cement
Pine Pressure		405	417.21	10	Ductile Iron
IJP1	10	374.92	374.92	4	Ductile Iron
EJP1	10	374.92	374.92	4	Ductile Iron
IJP2	10	374.92	374.92	4	Ductile Iron
EJP2	10	374.92	374.92	4	Ductile Iron
IPP1	15	405	405	8	Ductile Iron
EPP1	15	405	405	8	Ductile Iron
IPP2	15	405	405	8	Ductile Iron
EPP2	15	405	405	8	Ductile Iron

Pumps

P Name	P RIM	P Inv In	P Inv In 2	P Inv Out	P On	P Off	P Sump	P Power
Jean's Road Pump 1	52	374.92	374.92	374.92	373.5	372	381.32	10
Jean's Road Pump 2	52	374.92	374.92	374.92	374.5	372	381.32	10
Pine Street Pump 1	25	403.25	403.1	405	402.85	400.35	412.4	7.5
Pine Street Pump 2	25	403.25	403.1	405	403.85	400.35	412.4	7.5

Wet Wells

WW Name	W Ground	W Sump	W Inf	W Inf 2	P On	P Off
J Wet Well	381.32	370	374.92	374.92	373.5	372
P Wet Well	412.4	398	403.25	403.1	402.85	400.35

Pressure Junctions

PJ Name	PJ Elev
JJ	381.32
PJ	412.4

Outlet

O Name	O RIM	O Inv In	O Inv Out	O Sump
IPS	397.4	382	382	382

B.3 Flow Monitors

This section supplements Section 3.2.5 of Chapter 3.

Table B.3a contains the summaries of the raw data for the flow monitors installed in the field in downtown Veneta. The sensors operated from around 2:00 pm on February 1st to around 3:30 pm on February 19th. Measurements were recorded every five minutes, and then averaged over each hour. The flow was calculated by the sensor at each instant from velocity and fluid level measurements. Minimums, maximums and averages were located in the data, and the population for each region measured was factored in to find the data summarized in Table B.3a. The regions directly upstream of these sensors are green in the map in Section B.1 of this Appendix.

Table B.3a

Flow Monitoring Results			
Shed 4			
Avg Day/Cap	182.5	71,176.8	Average Daily Flow (GPD)
Max Day/Cap	528.5	206,130.7	Max Day (GPD)
Min Day/Cap	83.3	32,471.9	Min Day (GPD)
Max Hr/Cap	3,509.0	390.0	Population
		94.8	Dry Weather Base Flow (GPCPD)
		2.2	Peak Factor Average Day
		6.3	Peak Factor Max Day
Peak Hour	1,368,508.3	42.1	GPD = Max Hour
Shed 1, Subshed 2			
Avg Day/Cap	69.3	22,879.6	Average Daily Flow (GPD)
Max Day/Cap	221.5	73,081.7	Max Day (GPD)
Min Day/Cap	13.2	4,355.4	Min Day (GPD)
		330.0	Population
		16.2	Dry Weather Base Flow (GPCPD)
		5.3	Peak Factor Average Day
		16.8	Peak Factor Max Day
Peak Hour	104,000.0	23.9	GPD = Max Hour
Shed 2, Subshed 2			
Avg Day/Cap	128.6	23,014.1	Average Daily Flow (GPD)
Max Day/Cap	489.8	87,670.2	Max Day (GPD)
Min Day/Cap	46.0	8,241.5	Min Day (GPD)
		179.0	Population
		45.5	Dry Weather Base Flow (GPCPD)
		2.8	Peak Factor Average Day
		10.6	Peak Factor Max Day
Peak Hour	130,162.0	15.8	GPD = Max Hour

B.4 Flow Calculations

This section supplements Section 3.2.5 of Chapter 3.

The hydraulic model can only run when loads are applied at the manholes. At every manhole with a load, the load at a particular hour was calculated by multiplying together a population constant (2.89), the number of lots, the dry-weather base load (70 GPCPD), and a pattern constant based on the current hour. This section shows how we arrived at the flow pattern and base flow for the hydraulic model.

Base Flow The base flow of 70 GPCPD came from one of the driest days on record (February 23rd, 2008). We divided the total number of gallons received at the plant on that day (around 0.3 MGD) by the population served by the sewer (4300). Upon comparison to the results of flow monitors (see Table B.3a) it was determined that the resulting 70 GPCPD was appropriately conservative to use as the base load on every manhole in the model.

Flow Pattern The model's flow pattern, shown in Table B.2a and Table B.4a, was determined by studying the flow patterns observed at the wastewater treatment plant on the wettest day (January 6th, 2008) and the day with the highest hour (January 5th, 2008).

First, the hours with the highest volume on each day were combined into one hypothetical day. Then the flows at each hour of the combined day were raised by 10% in order to portray a more conservative storm event. The result was a hypothetical 'conservative day' with certain flow rates at each hour of the day.

Second, the flows at each hour of the conservative day were divided by the sewered population of Veneta (4300), then by the dry weather base flow (70) in order to find the load multiplier for each hour of the conservative day, as depicted in Table B.4a on the next page.

Table B.4a

Conservative Day Calculations								
Hour	**Jan 6 MGD**	**Jan 6 GPD**	**Jan 5 MGD**	**Jan 5 GPD**	**Combined Day GPD**	**Conserv-ative Day GPD (Combined x 110%)**	**Conserv-ative Day GPCPD**	**Factor**
0:00	2.38	2382000	1.80	1798900	2382000	2620200	602	8.60
1:00	2.42	2415900	1.73	1724800	2415900	2657490	610	8.72
2:00	2.30	2299600	1.74	1736000	2299600	2529560	581	8.30
3:00	2.17	2173100	1.70	1700700	2173100	2390410	549	7.84
4:00	2.05	2049100	1.66	1657800	2049100	2254010	518	7.40
5:00	2.00	2003900	1.74	1739400	2003900	2204290	506	7.23
6:00	2.00	2002800	1.76	1759200	2002800	2203080	506	7.23
7:00	1.90	1900200	1.73	1732700	1900200	2090220	480	6.86
8:00	1.86	1856200	1.77	1767700	1856200	2041820	469	6.70
9:00	1.86	1861400	1.76	1755800	1861400	2047540	470	6.72
10:00	1.91	1905400	1.73	1728700	1905400	2095940	481	6.88
11:00	1.92	1918200	1.81	1807500	1918200	2110020	485	6.92
12:00	2.06	2055600	2.06	2058000	2055600	2261160	519	7.42
13:00	2.18	2182200	2.68	2681800	2681800	2949980	678	9.68
14:00	2.30	2295900	2.63	2628400	2628400	2891240	664	9.49
15:00	2.29	2294700	2.47	2468500	2468500	2715350	624	8.91
16:00	2.29	2294000	2.40	2395800	2395800	2635380	605	8.65
17:00	2.23	2233600	2.30	2304000	2304000	2534400	582	8.32
18:00	2.20	2199300	2.35	2346300	2346300	2580930	593	8.47
19:00	2.17	2172500	2.41	2413400	2413400	2654740	610	8.71
20:00	2.11	2112000	2.45	2449400	2449400	2694340	619	8.84
21:00	2.17	2171300	2.44	2443700	2443700	2688070	617	8.82
22:00	2.21	2212800	2.41	2407700	2407700	2648470	608	8.69
23:00	2.25	2252000	2.35	2351900	2351900	2587090	594	8.49
24:00	2.38	2382000	1.80	1798900	2382000	2620200	602	8.60

B.5 Model Inconsistencies

This section discusses the manholes predicted by the model to overflow during the peak hour of the design load, as discussed in Section 3.4 of Chapter 3.

The two manholes that were observed to overflow each have a different cause. The manhole near the treatment plant, (MH 301 on Figure B1 in Section B.1 of this Appendix), floods when the surge basin at the treatment plant gets full. The water level, when spilling over the weir when the surge basin isn't full, is approximately six inches over the top of the weir, which is not quite enough to cause the nearby manhole to overflow. However, when the surge basin gets full, the level over the weir increases to approximately twelve or more inches, and this allows the nearby manhole to overflow.

The model does not simulate a surge basin, however it does simulate a submerged outfall with a maximum elevation (in the model this is called the 'tailwater elevation'). So, when the tailwater elevation is increased to be about twelve inches over the elevation of the weir at the plant, the model shows flooding. Also, when the tailwater elevation is only six inches over the weir, the model concurs with field observations, and does not predict flooding.

The manhole on Bolton Hill Road just east of Territorial (MH T-D on the map in Section B.1 of this Appendix) overflows into the street when there is heavy precipitation. Applying the available recorded peak hour flow to this region does not cause any flooding, which is consistent with observations by Public Works employees. When the designed peak hour is applied to the model, manholes MH T-D, followed by manholes MH Ba-3 and MH Ba-F, are predicted to overflow. This means the designed peak hour storm input (which is 10% greater than the greatest recorded flows at the plant) is sufficient to simulate conditions worse than those observed in the field.

B.6 2030 Loads on the 2007 Collection System

This section shows the map of the 2030 estimated loads modeled onto the 2007 unimproved collection system referred to in Section 3.5 of Chapter 3. A map of the results is found in Figure B2.

B.7 NE Study Analysis

This section discusses the Northeast Analysis referred to by Section 3.6 and its sub-sections in Chapter 3. This section also contains maps of the region and a profile of a gravity pipe that will serve the region in lieu of a small pump station.

The Northeast section of Veneta is an area inside the Urban Growth Boundary East of Territorial Highway and North of Highway 126 (depicted in Figures B3 and B4). This area is about 187 acres, and is currently serviced by a single 8 inch trunk that flows west to the West Lane center pump station and an individual service pressure system that provides sewer service for the new Cornerstone Employment Center, the Lane County facilities and the church. A separate alternative analysis for this area was conducted to determine how service could be provided to the Northeast Study area in the least expensive and most environmentally sensitive manner. Key components of this analysis require determination of:

- Which parts of the Northeast section of Veneta require pressure service?
- What type of pressure service is required?
- Where would lift stations be located?
- Which portion can be served using gravity?

The answers to these questions are displayed in Figures B3 and B4. The analysis leading to these answers is discussed below.

Figure B3

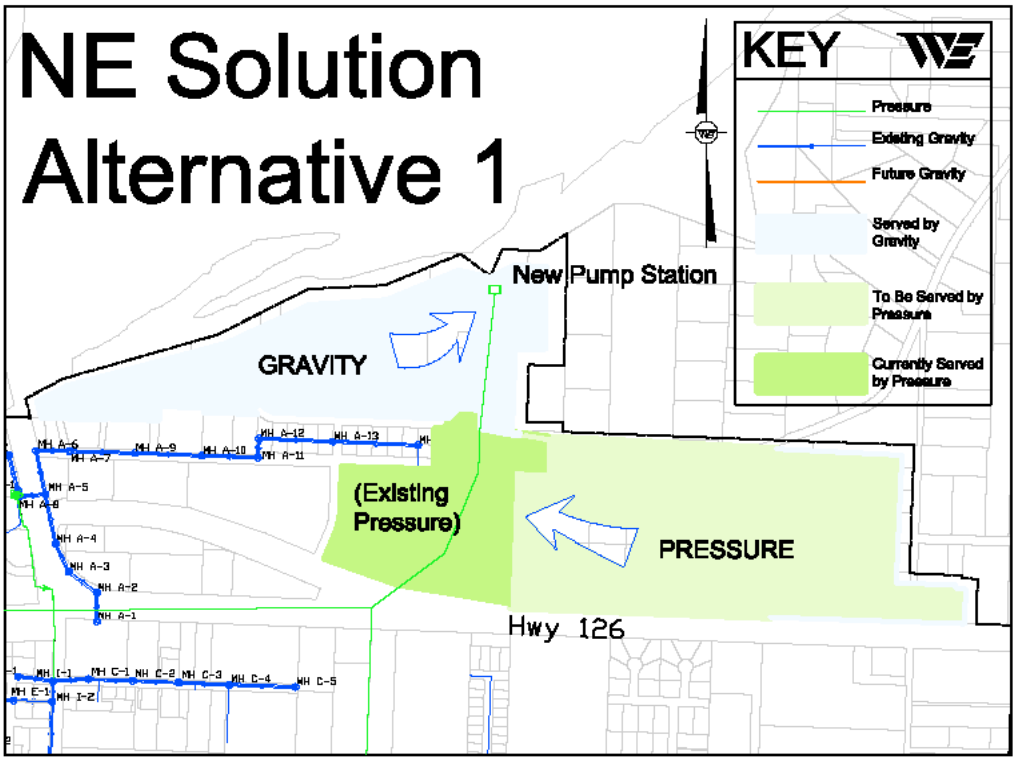
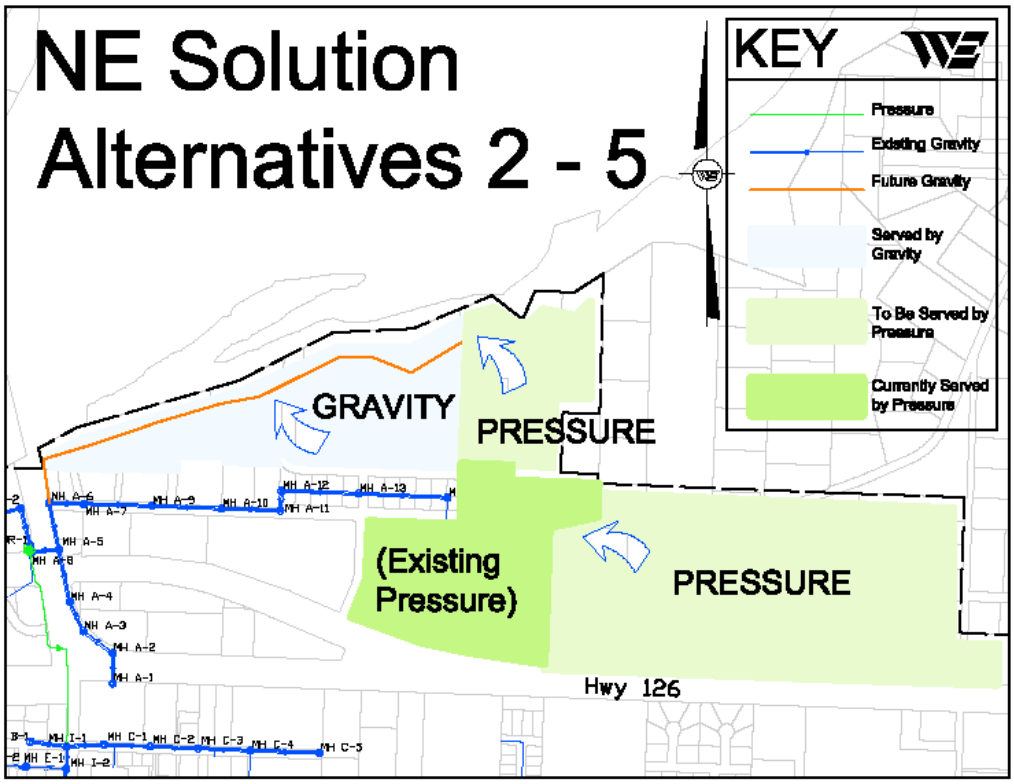


Figure B4



NE Analysis

The topography of the Northeast section of Veneta was evaluated and low points in the terrain identified. Routes for gravity pipes that could be buried at least three feet deep were compared for their construction costs and environmental impact (see Figure B5 below). As a rule, the shallower the sewer, the less expensive and environmentally disruptive it is to construct. Remaining areas that were too low for gravity service to be feasible would be served by individual pumps. The following scenarios were developed for each alternative:

- ***Alternative NE-1*****,** depicted by Figure B3, requires a pump station to serve the northern-most section of the Northeast. This allows for the area immediately surrounding the new pump station (north of Jean's Road) to be served by gravity. The existing area requiring individual pumps would be expanded eastward to Huston.
- ***Alternatives NE-2—NE-5***, depicted by Figure B4, require the area north of Jean's Road in the Northeast to be served mostly by a future gravity line connecting to the existing 8 inch trunk that empties to West Lane pump station. Figure B5 provides details on the gravity lines in this area. The low area immediately to the east of this gravity line would require individual pump stations for service. The existing area requiring individual pumps would also be expanded eastward to Huston.

These NE scenarios represent the least capital and environmental cost solution to service in this area. The deeper the sewer and pump station, the more expensive the capital cost. Our analysis originally recommended a crossing underneath Highway 126 and the railroad, so as to provide additional gravity service to the Northeast. However, due to excessive costs associated with construction underneath Highway 126 and the railroad, this recommendation was abandoned. This will result in a slightly shallower gravity network near the new Huston pump station, which will further decrease construction costs and environmental impact in that area.

B.8 Alternative Cost Comparisons

This section presents the breakdown of construction cost estimates for each collection system alternative, referred to in Section 3.8.3 of Chapter 3. The breakdowns for Alternatives 1, 2 and 3 are found in Tables B.8a, B.8b and B.8c, respectively. The breakdown of long term operations and maintenance cost estimates for each alternative is found in Table B.8d.

Figure B5

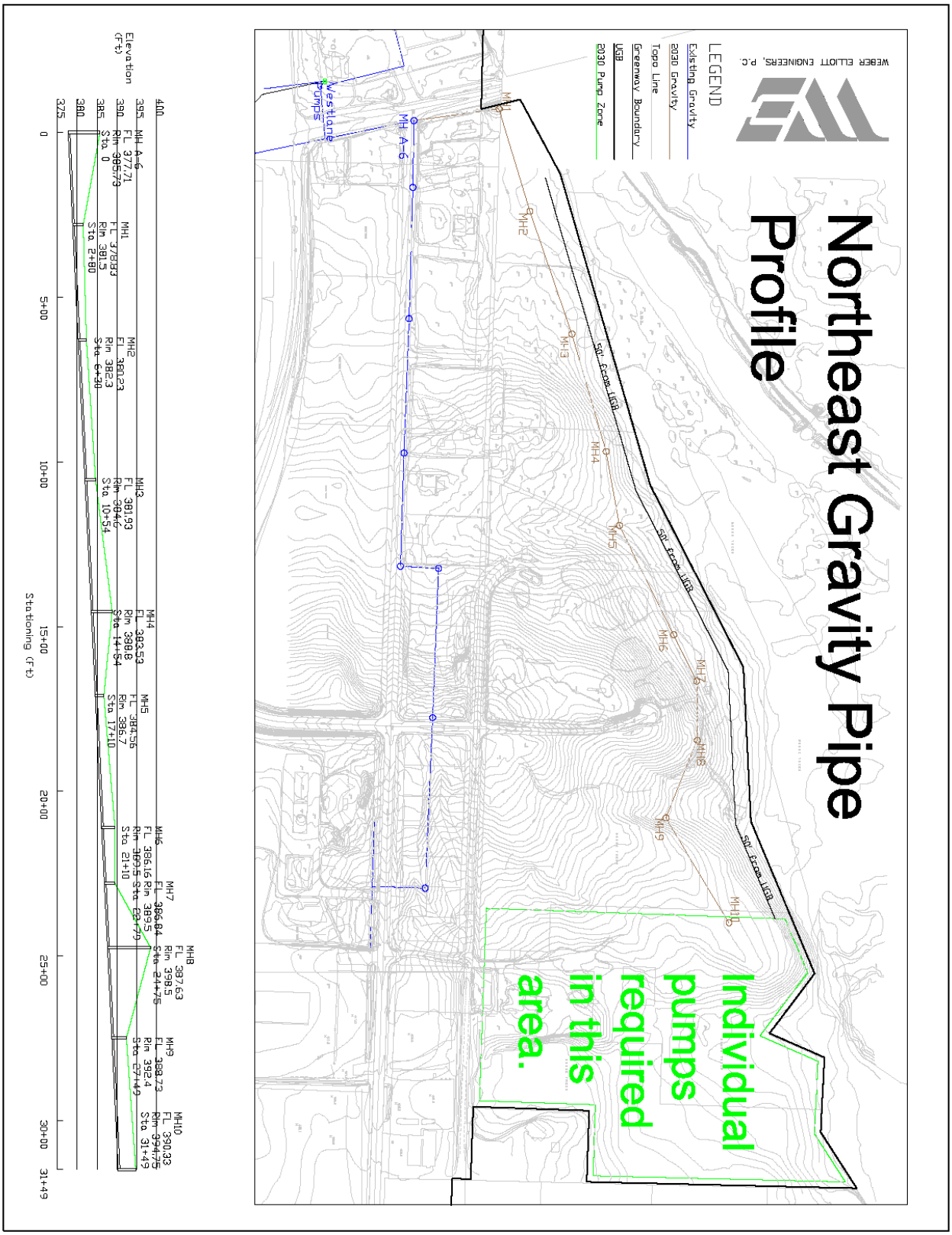

Northeast Gravity Pipe Profile
WEBER ELLIOTT ENGINEERS, P.C.
LEGEND
Existing Gravity
2030 Gravity
Topo Line
Greenway Boundary
UGB
2030 Pump Zone
Individual pumps required in this area.
Westlane Pumps
MH A-6
MH1
MH2
MH3
MH4
MH5
MH6
MH7
MH8
MH9
MH10
Sgl Ecoa UGB
Elevation (Ft)
400
395
390
385
380
375
0
5+00
10+00
15+00
20+00
25+00
30+00
31+49
Stationing (Ft)
MH A-6 FL 377.71 Rim 385.73 Sta 0
MH1 FL 378.87 Rim 381.5 Sta 2+80
MH2 FL 381.23 Rim 382.3 Sta 6+30
MH3 FL 381.93 Rim 384.6 Sta 10+54
MH4 FL 383.53 Rim 388.8 Sta 14+54
MH5 FL 384.56 Rim 386.7 Sta 17+10
MH6 FL 386.16 Rim 389.5 Sta 21+10
MH7 FL 386.84 Rim 389.5 Sta 21+75
MH8 FL 387.63 Rim 398.5 Sta 24+75
MH9 FL 388.73 Rim 392.4 Sta 27+49
MH10 FL 390.33 Rim 394.75 Sta 31+49

Table B.8a

Alternative 1 Costs					
Project	**Element**	**Description**	**Units**	**QTY**	**Total Cost**
1	1	Install new 12" pressure line from Waste Water Treatment Facility to Westlane forcemain	feet	5,766	$306,800
2	2	Install new 12" pressure line from Westlane forcemain to Hwy 126 near railroad	feet	2,094	$101,400
3	3	New North Pump Station north of Todd Way near shore of Fern Ridge Resevoir	each	1	$277,800
4	4	Install new 4" forcemain for New North Pump Station	feet	2,374	$68,300
5	5	Install new 12" pressure line from Hwy 126 near railroad to Hunter	feet	1,430	$97,700
6	6	New East Pump Station 800 feet south of Hunter off Huston	each	1	$333,600
7	7	Install new 8", 12" gravity from New East Pump Station north to Tidball and west to Fern Meadow	feet	2,790	$426,800
8	8	Install new 8" forcemain for New East Pump Station	feet	3,598	$120,300
9	9	Install new 12" forcemain extension for Pine Street Pump Station	feet	454	$23,900
10	10	Install new 8" forcemain extenstion for New East Pump Station	feet	814	$27,200
11	11	Upsize to new 21" modified gravity pipe under Hunter/Territorial Intersection	feet	173	$190,000
12	12	Upsize to new 10", 12", 15" gravity pipe under Territorial south of Hunter from MH I-5 to MH Ba-6	feet	2,115	$374,000
13	13	Common Projects: upsize gravity pipe under Parkside, 8th, Cheney and Oak Island from 8" to 10" or 12"	feet	1,564	$248,400
14	14	Air & Vacuum Release Valves	each	5	$25,000
15	15	Replace pumps at Westlane station	each	2	$22,500
16	16	Replace pumps at Pine Street station	each	2	$41,200
17	17	Main Pump Control Center (SCADA Brain)	each	1	$32,000
18	18	Land Acquisition	sq.ft.	9,000	$90,000
19	19	Easements, Right-of-Way	feet	1,680	$35,300
				Construction Total:	**$2,842,200**
				Design & Engineering:	**$568,400**
				Contingencies:	**$568,400**
				Legal & Administrative:	**$142,000**
				Total Capital:	**$4,121,000**

Table B.8b

Alternative 2 Costs					
Project	**Element**	**Description**	**Units**	**QTY**	**Total Cost**
1	1	Upsize to 12", 15", 21" gravity pipe under Territorial Hwy from MH I-3a to MH Ba-2	feet	787	$182,000
2	2	Upsize to new 21" modified gravity pipe under Hunter/Territorial Intersection	feet	173	$190,000
3	3	Upsize to 10" gravity pipe under Territorial Hwy from MH Ba-2 to MH Ba-6	feet	1,475	$227,300
4	4	Upsize to 21" gravity pipe from MH I-4 to MH I-11 along Hunter	feet	1,769	$678,700
5	5	Upsize to 21" gravity pipe from MH N-7 to MH 303 along Hunter	feet	1,547	$336,000
6	6	Install new 21" gravity pipe bypass from MH I-11 to MH N-7 along Hunter	feet	754	$177,900
7	7	Common Projects: upsize gravity pipe under Parkside, 8th, Cheney and Oak Island from 8" to 10" or 12"	feet	1,564	$248,400
8	8	New East Pump Station on Huston Rd. south of Hunter	each	1	$330,000
9	9	Install new 8" force main for New East Pump Station	feet	3,612	$120,800
10	10	Continue 8" force main for New East Pump Station	feet	1,306	$43,700
11	11	Install new 12" gravity line from New East wet well	feet	1,100	$196,300
12	12	Install new screw pump at treatment facility	each	1	$284,200
13	13	Replace pumps at Westlane station	each	2	$14,400
14	14	Replace pumps at Pine Street station	each	2	$33,400
15	15	Air & Vacuum Release Valves	each	2	$10,000
16	16	Land Acquisition	sq.ft.	6,000	$60,000
17	17	Private property easement	feet	682	$14,300

Construction Total:	**$3,147,400**
Design/Engineering:	**$629,480**
Contingencies:	**$629,480**
Legal/Administrative:	**$157,400**
	$4,563,760

Table B.8c

Alternative 3 Costs					
Project	**Element**	**Description**	**Units**	**QTY**	**Total Cost**
1	1	Upsize to 18" gravity pipe under Territorial Hwy from MH I-4 to MH I-5	feet	63	$23,200
2	2	Upsize to 18" gravity pipe under Hunter from MH I-4 to MH I-9	feet	1,156	$371,600
3	3	Upsize to 21" gravity pipe under Hunter from MH M-7 to MH 303	feet	1,547	$336,000
4	4	Install new 21" gravity pipe bypass from MH I-11 to MH M-7 along Hunter	feet	754	$177,900
5	5	Upsize to 10" gravity pipe under Territorial Hwy from MH Ba-F to MH Ba-6	feet	1,328	$205,300
6	6	Install new 12", 15" gravity bypass under Woodland from MH Ba-F to MH I-13	feet	2,210	$472,200
7	7	Install new 8" gravity bypass under 6th from MH 6-B to MH N-2	feet	91	$15,000
8	8	Install new 10" gravity bypass under 6th from MH H2-1, crossing Hunter, to MH N-1	feet	656	$151,000
9	9	Upsize to 10" gravity pipe under 6th from MH H2-6 to MH H2-1	feet	1,265	$218,000
10	10	Install new 6" pressure bypass under Waldo from Westlane force main to MH H2-6	feet	1,237	$24,300
11	11	Common Projects: upsize gravity pipe under Parkside, 8th, Cheney and Oak Island from 8" to 10" or 12"	feet	1,564	$248,400
12	12	New East Pump Station on Huston Rd. south of Hunter	each	1	$330,000
13	13	Install new 8" force main for New East Pump Station	feet	3,612	$120,800
14	14	Continue 8" force main for New East Pump Station	feet	1,306	$43,700
15	15	Install new 12" gravity line from New East wet well	feet	1,100	$196,300
16	16	Install new screw pump at treatment facility	each	1	$284,200
17	17	Replace pumps at Westlane station	each	2	$14,400
18	18	Replace pumps at Pine Street station	each	2	$33,400
19	19	Air & Vacuum Release Valves	each	2	$10,000
20	20	Land Acquisition	sq.ft.	5,020	$50,200
21	21	Private property easement	feet	682	$14,300

Construction Total:	**$3,340,200**
Design/Engineering:	**$668,000**
Contingencies:	**$668,000**
Legal/Administrative:	**$167,000**
Total Capital:	**$4,843,200**

Table B.8d

Alternative 5 Costs					
Project	**Element**	**Description**	**Units**	**QTY**	**Total Cost**
1	1	Upsize to 12", 15" gravity pipe under Territorial Hwy from MH I-5 to MH Ba-2	feet	751	$165,600
2	2	Upsize to 10" gravity pipe under Territorial Hwy from MH Ba-2 to MH Ba-6	feet	1,475	$227,300
3	3	Install new 24" gravity pipe from MH 303 to MH N-2 by the treatment facility	feet	436	$87,000
4	4	Install new 21" gravity pipe from MH N2 to MH N9 along Dunham and from MH N9 to MH I-3 along McCutcheon	feet	4,315	$1,258,900
5	5	Install new clean-out on 7th, 6th, 5th, 4th, 3rd and Territorial	each	6	$24,000
6	6	Install new 21" gravity pipe from MH I-3 to I-5 on Territorial, re-routing all flows from the South or East	feet	311	$91,500
7	7	Common Projects: upsize gravity pipe under Parkside, 8th, Cheney and Oak Island from 8" to 10" or 12"	feet	1,564	$248,400
8	8	New East Pump Station on Huston Rd. south of Hunter	each	1	$330,000
9	9	Install new 8" force main for New East Pump Station	feet	3,612	$120,800
10	10	Continue 8" force main for New East Pump Station	feet	1,306	$43,700
11	11	Install new 12" gravity line from New East wet well	feet	1,100	$196,300
		Install new screw pump at treatment facility	each	1	$284,200
12	12	Replace pumps at Westlane station	each	2	$14,400
13	13	Replace pumps at Pine Street station	each	2	$33,400
14	14	Air & Vacuum Release Valves	each	2	$10,000
15	15	Private property easement	feet	830	$17,400
16	16	Permits	sq.ft	11,600	$46,400

Construction Total:	**$3,199,300**
Design/Engineering:	**$639,900**
Contingencies:	**$639,900**
Legal/Administrative:	**$160,000**
Total Capital:	**$4,639,100**

Table B.8e

PROJECTED PUMP OPERATING COSTS			
Alternative 1		**Alternative 2**	
Screw Pump:	**$126,300**	Screw Pump:	**$195,900**
Power:	**$82,100**	Power:	**$74,300**
Operations:	**$125,700**	Operations:	**$99,800**
Maintenance:	**$286,900**	Maintenance:	**$221,700**
Total:	**$621,000**	**Total:**	**$591,700**
Alternative 3		**Alternative 5**	
Screw Pump:	**$195,900**	Screw Pump:	**$195,900**
Power:	**$80,200**	Power:	**$74,300**
Operations:	**$99,800**	Operations:	**$99,800**
Maintenance:	**$221,700**	Maintenance:	**$221,700**
Total:	**$597,600**	**Total:**	**$591,700**

Cost Estimates

Estimates were made for construction materials and labor costs. The prices are listed below in Table B.8f:

Table B.8f

Construction Cost Estimates			
Item	**Unit Cost**		**Unit**
Fuel Price	$5.00	per	Gallon
Truck Driver	$41.01	per	Hour
Truck	$35.00	per	Hour
Asphalt	$75.00	per	Ton
Loose Aggregate	$16.00	per	Ton
10" PVC Pipe	$5.88	per	Linear Foot
12" PVC Pipe	$8.43	per	Linear Foot
15" PVC Pipe	$12.50	per	Linear Foot
18" PVC Pipe	$19.37	per	Linear Foot
21” PVC Pipe	$46.02	per	Linear Foot
24" PVC Pipe	$60.00	per	Linear Foot
6" Class 200 Pressure Pipe	$4.80	per	Linear Foot
8" Class 200 Pressure Pipe	$8.40	per	Linear Foot
10" Class 200 Pressure Pipe	$12.40	per	Linear Foot
12" Class 200 Pressure Pipe	$17.60	per	Linear Foot
Shoring	$1.50	per	Square Foot
Pavement Cut	$1.30	per	Linear Foot
Excavation	$14.00	per	Cubic Yard
Flagger Rate	$34.23	per	Hour
Manhole	$3,500.00	per	Manhole
Lateral – 4” installation	$645.00	per	Lateral
Lateral – 4” replacement	$2,700.00	per	Lateral
Hose Rental	$10.00	per	Day
Trash Pump Rental	$51.00	per	Day

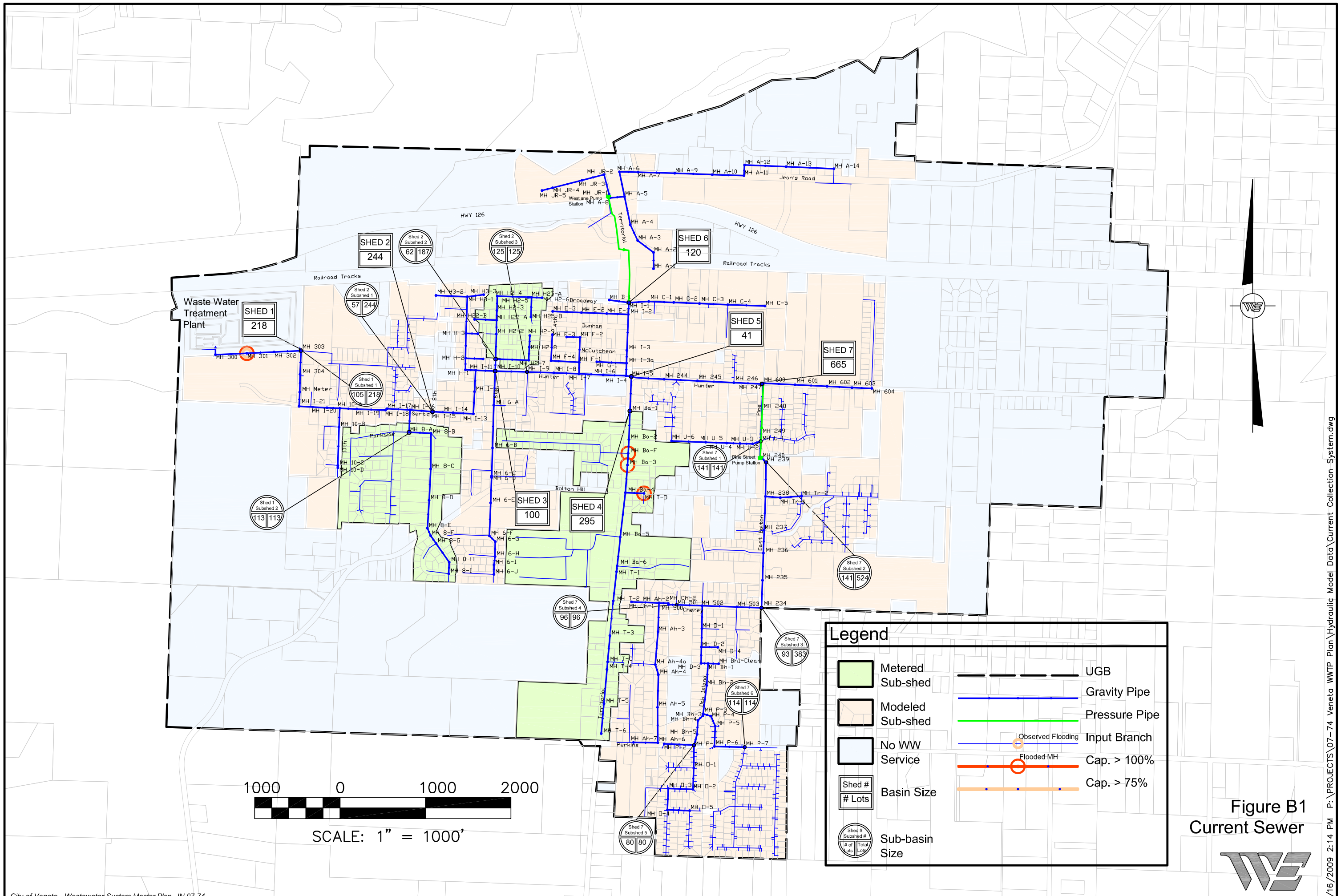


Figure B1
Current Sewer

WEBER ELLIOTT ENGINEERS, P.C.

City of Veneta - Wastewater System Master Plan JN 07-74

1/16/2009 2:14 PM P:\PROJECTS\07-74 Veneta WWTP Plan\Hydraulic Model Data\Current Collection System.dwg

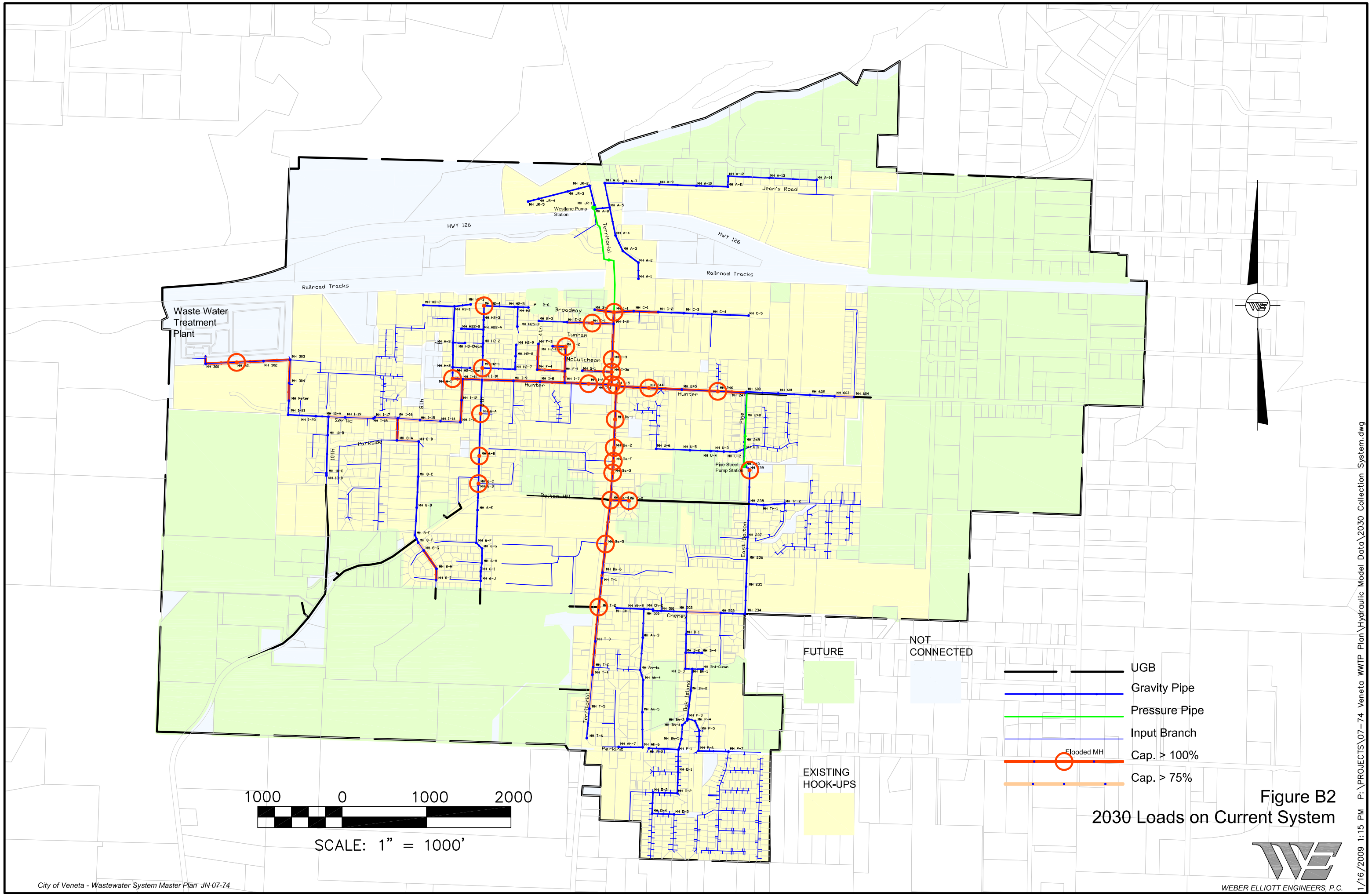


Figure B2
2030 Loads on Current System

APPENDIX C

Weber Elliott Engineers, P.C.
JTE
Created 8/12/2008
Job No. 07-74

TABLE C-1: TREATMENT TRAIN ESTIMATE OF COST

City of Veneta

Wastewater Facility Planning

Revised: JTE 3/19/2009

ENGINEER'S ESTIMATE

Item	Quantity	Unit	Price per unit	Total
Headworks and Screening				
Headworks Hydraulic Analysis Report	1	LS	$40,000.00	$40,000
Flow Meassure				
Hunter Road Flow Measure with SCADA	1	LS	$24,000.00	$24,000
Collection System Flow Measure	1	LS	$15,000.00	$15,000
Mobilization/Site Preparation	1	LS	$8,000.00	$8,000
Pump Modifications at Plant				
Screen Cover for screws	3	LS	$11,800.00	$35,400
Mobilization/Site Preparation	1	LS	$6,000.00	$6,000
Excavation	177	CY	$33.00	$5,841
Speciality changes - gates etc	1	LS	$22,000.00	$22,000
Concrete Flight	13	CY	$1,100.00	$14,300
Screw Pump	1	EA	$68,000.00	$68,000
Wiring/Control Integration	1	LS	$15,600.00	$15,600
Surge Basin Changes				
None scheduled				
Screening				
Mobilization/Site Preparation	1	L.S.	$6,000.00	$6,000
Demolish Existing Structure	23	CY	$200.00	$4,600
Remove existing wiring/plumbing	1	LS	$800.00	$800
Pair Step Screen 6MM	2	each	$68,800.00	$137,600
New Channel Concrete	33	CY	$1,100.00	$36,300
Wiring, Replumbing Mount	1	LS	$12,000.00	$12,000
Bypass Facilities	1	LS	$9,000.00	$9,000
Flow Splitting Structure				
Demolish Existing Structure	23	CY	$200.00	$4,600
Remove Plumbing / wiring	1	each	$400.00	$400
			Subtotal: Construction Costs:	$465,441
			Design Engineering:	$93,088
			Contingency:	$93,088
			Legal and Administrative:	$23,272
Total				**$674,900**
Biolac Expansion				
Remove Exist. FSL Liner	4530	Sq. Yds	$1.50	$6,795
Demo Exist Conc. Weir + Access Ramps	57	CY	$200.00	$11,400
Excavation	54	cy	$25.00	$1,350
Hauling	54	CY	$12.00	$648
Fill (Imported Fill)	8120	cy	$29.60	$240,352
Clarifier / RAS Concrete	209	cy	$1,100.00	$229,900

APPENDIX C

TABLE C-1: TREATMENT TRAIN ESTIMATE OF COST

City of Veneta

Wastewater Facility Planning

Revised: JTE 3/19/2009

ENGINEER'S ESTIMATE

Item	Quantity	Unit	Price per unit	Total
Miscellaneous Access concrete	26	cy	$700.00	$18,361
Parkson August '08 Quote	1	LS	$514,000.00	$514,000
Aerator Upgrades	3	EA	$8,500.00	$25,500
Shipping from Florida	1	LS	$14,000.00	$14,000
Install Piping, assembly, off loading etc.	1	LS	$154,000.00	$154,000
Install Electrical and Controls	1	LS	$40,000.00	$40,000
Access Road and Site Pavement	80	TON	$90.00	$7,200
Air Piping building to Basins	390	lf	$110.00	$42,900
RAS Piping Valves & Misc	1	LS	$21,000.00	$21,000
Control Building Changes	1	LS	$20,000.00	$20,000
Basin Liner (prep, liner, cushion, etc.)	16500	Sq. Ft.	$3.65	$60,225
			Subtotal: Construction Costs:	$1,407,631
			Design Engineering:	$281,526
			Contingency:	$281,526
			Legal and Administrative:	$70,382
Total				**$2,041,100**
Modify FSL				
Excavation	2111	cy	$25.00	$52,775
Fill (Imported Fill)	3540	Cy	$29.60	$104,784
Liner	37000	Sq. Ft.	$3.65	$135,050
Misc Structures (Concrete)	57	Cy	$1,100.00	$62,700
Pipe	1200	L.F	$110.00	$132,000
Access Road Pavement	225	TON	$90.00	$20,250
			Subtotal: Construction Costs:	$507,559
			Design Engineering:	$101,512
			Contingency:	$101,512
			Legal and Administrative:	$25,378
Total				**$736,000**
UV System Changes (2009 Requirements)				
Excavation	28	CY	$25.00	$700
Hauling	28	CY	$12.00	$336
Fill (Imported Fill)	4	CY	$29.60	$118
Misc Structures (Concrete)	13	CY	$1,100.00	$14,300
Pipe From New Biolac Basin	270	L.F	$110.00	$29,700
Access Road Pavement	14	TON	$90.00	$1,260
Equipment	1	LS	$160,400.00	$160,400
Shipping and handling	1	LS	$8,400.00	$8,400
Installation - Hydraulic	1	LS	$12,000.00	$12,000
Installation - Labor - Lamps	100	Hrs	$72.00	$7,200
Installation electrical control Power upgrade	1	LS	$23,000.00	$23,000

APPENDIX C

TABLE C-1: TREATMENT TRAIN ESTIMATE OF COST

City of Veneta

Wastewater Facility Planning

Revised: JTE 3/19/2009

ENGINEER'S ESTIMATE

Item	Quantity	Unit	Price per unit	Total
Weather Protection/cover	1	LS	$12,300.00	$12,300
			Subtotal: Construction Costs:	$269,714
			Design Engineering:	$53,943
			Contingency:	$53,943
			Legal and Administrative:	$13,486
Total				**$391,100**
Future UV System Changes (2017 Requirements)				
Misc Structures (Concrete)	2	CY	$1,100.00	$2,200
Equipment	1	LS	$160,400.00	$160,400
Shipping and handling	1	LS	$8,700.00	$8,700
Installation - Hydraulic	1	LS	$8,400.00	$8,400
Installation - Labor - Lamps	90	Hrs	$72.00	$6,480
			Subtotal: Construction Costs:	$186,180
			Design Engineering:	$37,236
			Contingency:	$37,236
			Legal and Administrative:	$9,309
Total				**$270,000**
Effluent Modifications / Upgrade				
Planting Program	18	Acres	$4,000.00	$72,000
Reuse Upgrade	1	L.S.	$5,000.00	$5,000
40 HP Motors - Irrigation	2	each	$4,700.00	$9,400
Installation Labor / Irrigation	80	Hrs	$72.00	$5,760
Pump Upgrades - Irrigation	2	each	$3,780.00	$7,560
Electrical Upgrades - Irrigation	1	LS	$7,000.00	$7,000
Platform for expanded fine filters - Irrigation	1	LS	$6,200.00	$6,200
Fine filters - installed	2	each	$12,300.00	$24,600
Long Tom River Outlet Hydraulic Changes	1	L.S.	$37,000.00	$37,000
Not Potable Water System Changes	1	L.S.	$17,400.00	$17,400
			Subtotal: Construction Costs:	$191,920
			Design Engineering:	$22,984
			Contingency:	$38,384
			Legal and Administrative:	$5,746
Total				**$259,000**

APPENDIX C

TABLE C-1: TREATMENT TRAIN ESTIMATE OF COST

City of Veneta

Wastewater Facility Planning

Revised: JTE 3/19/2009

ENGINEER'S ESTIMATE

Item	Quantity	Unit	Price per unit	Total
Grand total				**$4,372,100**

Note: All totals rounded to the nearest dollar.