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**City of Toledo**

Lincoln County, Oregon

# WASTEWATER FACILITIES PLAN

March 2014


Civil West Engineering Services, Inc.

Civil West Engineering Services, Inc. • 486 E Street • Coos Bay, Oregon 97420

# City of Toledo

LINCOLN COUNTY, OREGON

## Wastewater Facilities Plan

***March 2014***

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# 1.0 Executive Summary

Section

1

## *1.1. Background*

The City of Toledo owns and maintains a wastewater conveyance system that collects, transmits, and treats sanitary wastewater from residential and commercial customers within the City's system. Today, according to the 2010 Census data, the City of Toledo wastewater system provides sanitary service to approximately 3700 persons.

In 1954 the City of Toledo built the treatment plant, including several of the current lift stations and separated the sanitary and storm sewers systems. The original plant consisted of a primary clarifier, an anaerobic digester, an effluent metering station, an 18" outfall to the river, and sludge drying beds south of the railroad tracks. Currently the original primary clarifier and anaerobic digester are still in use as a secondary clarifier and sludge storage tank respectively, and the original 18" outfall is still in use.

In 1970, the City constructed a concrete contact stabilization package plant to provide secondary treatment capabilities. In 1981, the City doubled the treatment plant hydraulic capacity to 3.2 million gallons per day (mgd) with the addition of a headworks, a second contact stabilization unit, and a second final clarifier. In 1991 substantial improvements were made to the system which included upgrades to the lift stations, the collection system and the treatment plant. Most recently, in 2000, various units of the treatment plant were upgraded to increase treatment capacity.

The most recent Facilities Plan for the City wastewater facilities was prepared by Clearwater Engineering in 1993, which paired with a Wastewater Master Plan prepared in 1995. These resulted in the year 2000 improvements. The end of the 20 year planning period is quickly approaching and the City of Toledo wishes to have in place a new plan which identifies and addresses the current needs of the wastewater system and recommends specific upgrades to the wastewater systems.

The City's lift stations are showing their age and have experienced failures in recent years. While the City has worked hard to maintain these facilities, it is becoming increasingly difficult to provide reliable service with this aging infrastructure.

Considering the age of the existing Toledo Wastewater Facilities Plan and the condition and needs associated with the City's wastewater system, the time has come to complete a new wastewater facilities plan for Toledo.

## *1.2. Recommended Improvement Projects*

Due to the age and deficiency of portions of the City's wastewater system, we have evaluated numerous options for improvements. A summary of the final recommendations is below:

**Priority 1 Projects:**

- **Wastewater Treatment Facility Improvements:** It is recommended that the City construct improvements to remedy the wastewater treatment facility deficiencies. The upgrades to the treatment facility should include a number of components to improve operations of the facility as follows:
    - **Headworks:** Replace the flow equalization weir.
    - **New Effluent Booster Pumps:** Install new effluent booster pumps.
    - **New Outfall:** Replace a portion (~300') of the outfall pipe.

- **Sludge Handling and Storage:** Install a new sludge holding tank to free up both treatment units.

- **Lift Station Improvements:** The next Priority 1 improvement projects involve completing improvements necessary at the City's Wastewater Lift Stations. The following series of improvement projects are recommended at the following lift stations:
  - **Butler Bridge Lift Station Improvements:** Reconstruct Butler Bridge Lift Station to use submersible pumps in lieu of a wetwell/drywell configuration.

  - **Butler Bridge Lift Station Force Main:** As part of the Butler Bridge Lift Station upgrades, it is also recommended that the old portion (~1100 ft) of the existing force main be replaced with a new 14-inch force main.

  - **Ammon Road Lift Station Improvements:** Reconstruct Ammon Road Lift Station to use submersible pumps in lieu of a wetwell/drywell configuration.

- **Gravity Collection System Improvements:** The final Priority 1 projects identified involve completing necessary improvements to the City's gravity wastewater collection system. These improvements were identified and prioritized in the I&I investigation report which is provided in Appendix C. Below is a general description of the type of improvements required:

  - **Pipe Improvements:** Improvements to the gravity systems existing collection pipes include: pipe replacement, lining, pipe bursting, and pipeline patches. For a more detailed breakdown of the proposed improvements and their locations within the collection system please refer to the I&I study provided in Appendix C.

  - **Manhole Improvements:** Improvements to the gravity systems existing manholes include: replacement, lining, patching, and grouting of the systems manholes. For a more detailed breakdown of the proposed improvements and their locations within the collection system please refer to the I&I study provided in Appendix C.

**<u>Priority 2 Projects:</u>**

- **Lift Station Improvements:** The following series of projects have been identified as Priority 2 projects and are located at the following lift stations:

  - **"A" Street Lift Station Improvements:** Basic improvements are recommended for the "A" Street Lift Station including upgrading piping, pumps, fittings, structural upgrades, electrical and control systems. The upgrades are intended to extend the life of the facility and improve the operation and maintenance issues related to the pump station.

  - **"A" Street Lift Station Force Main:** As part of the "A" Street Lift Station upgrades, it is also recommended that the facilities existing force main be replaced with a new 12-inch force main.

- **Gravity Collection System Improvements:** The final Priority 2 projects identified involve completing necessary improvements to the City's gravity wastewater collection system. These improvements where identified and prioritized in the I&I investigation report which is provided in Appendix C.

**Priority 3 Projects:**

- **Lift Station Improvements:** The following series of improvement projects have been identified as Priority 3 projects and are located at the following lift stations:

    - **High School Lift Station Improvements:** Basic upgrades are recommended for the High School Lift Station. Improvement recommendations include piping and fitting upgrades, generator installation, controls and electronic upgrades and structural upgrades. These recommendations are intended to extend the useful life of the pump station through and beyond the planning period.

    - **Lincoln Way Lift Station Improvements:** Basic upgrades are recommended for the Lincoln Way Lift Station. Improvement recommendations include piping and fitting upgrades, generator installation, controls and electronic upgrades and structural upgrades. These recommendations are intended to extend the useful life of the pump station through and beyond the planning period.

- **Gravity Collection System Improvements:** The final Priority 3 improvement projects identified involve completing necessary improvements to the City's gravity wastewater collection system. These improvements where identified and prioritized in the I&I investigation report as both priority level 3 and 4, a copy of the I&I is provided in Appendix C, but are combined into a single priority level for inclusion into this report.

## *1.3. Summary of Capital Improvement Plan and Funding*

**Table 1.3 - Recommended Project Cost Summary**

| Recommended Improvements and Alternatives: | | | |
|---|---|---|---|
| **Priority 1 Projects:** | | | |
| **Facility** | **Alternative, Recommendation** | **Description** | **Total Cost** |
| Wastewater Treatment Plant | Headworks | New Flow Equalization Weir | $25,000 |
| | Outfall Pipe | Replace Portion of Outfall | $207,230 |
| | Effluent Booster Pumps | Install Effluent Booster pumps | $246,935 |
| | Sludge Alternative A | Sludge Storage Tank | $514,829 |
| Ammon Road Lift Station | Alternative B | New Wet Well | $1,303,543 |
| Butler Bridge Lift Station | Alternative B | New Wet Well | $1,404,767 |
| Butler Bridge Force Main | Recommendation | Replace Portion of Force Main | $262,049 |
| Collection System (Piping and Manholes) | I & I - Priority 1 | Pipe Replacement, Lining, Bursting or Patching; Manhole Rehabilitation | $380,935 |
| | | Total Priority 1 Projects: | $4,345,288 |
| **Priority 2 Projects:** | | | |
| **Facility** | **Alternative, Recommendation** | **Description** | **Total Cost** |
| "A" Street Lift Station | Alternative A | Dry Pit Upgrade | $671,248 |
| "A" Street Lift Station Force Main | Recommendation | Replace Force Main | $172,175 |
| Collection System (Piping and Manholes) | I & I - Priority 2 | Pipe Replacement, Lining, Bursting or Patching; Manhole Rehabilitation | $565,400 |
| | | Total Priority 2 Projects: | $1,408,823 |
| **Priority 3 Projects:** | | | |
| **Facility** | **Alternative, Recommendation** | **Description** | **Total Cost** |
| High School Lift Station | Alternative B | Upgrades and Life Extension Improvements | $233,651 |
| Hospital Lift Station | Alternative B | Upgrades and Life Extension Improvements | $148,928 |
| Collection System (Piping and Manholes) | I & I - Priority 3 & 4 | Pipe Replacement, Lining, Bursting or Patching; Manhole Rehabilitation | $490,340 |
| | | Total Priority 3 Projects: | $872,919 |
| | | **Total Overall Plan Cost:** | **$6,627,030** |

The impact to rate payers of the recommended improvements is $17.49 per month for Priority 1 improvments, $6.34 per month for Priority 2 improvements and $4.98 per month for Priority 3 improvements. Given likely increases in operation and maintenance costs, the City should plan on rate increases of up to $29 over the next ten years. Given current rates, which average $61 per EDU, this represents a 48% increase and would increase the average rate to approximately $90 per month.

Section

2

# 2.0 Introduction, Background and Need

## 2.1. Background

The City of Toledo owns and maintains a wastewater conveyance system that collects, transmits, and treats sanitary wastewater from residential and commercial customers within the City's Urban Growth Boundary (UGB). Today, according to the 2010 Census data, the City of Toledo the population of the City was 3465 persons. The City's wastewater system provides sanitary service for up to 3700 persons (this includes the population within the city limits, the high school and others outside of the city limits but within the UGB).

The City of Toledo's sanitary sewer system was originally constructed in 1926 as a combined sanitary sewer and storm sewer system which discharged directly into the Yaquina River without any treatment. The first sewers were concrete bell and spigot pipe with mortared joints, some of which are still in service today.

The City built their original treatment plant in 1954 which included several of the current lift stations and the separation of sanitary and storm sewers. The original plant consisted of a primary clarifier, an anaerobic digester, an effluent metering wier, an 18" outfall to the river, and sludge drying beds located south of the railroad tracks. Currently, the original primary clarifier and anaerobic digester are still in use as a secondary clarifier and sludge storage tank respectively, and the original 18" outfall is still in use. The original effluent structure and sludge drying beds have been abandoned.

The original system was designed to allow a portion of the peak flows to overflow into the sloughs from the lift stations whenever the pump capacities were exceeded.

In the late 1960's, the City identified that the major factor in overflows was due to infiltration/inflow associated with the old pipes and began to replace sections of the original piping.

In 1970, the City upgraded the treatment plant by constructing and integrating a concrete contact stabilization package plant with the existing facilities to provide secondary treatment capabilities. Also included in this upgrade was an enhancement of the existing chlorine disinfection system.

In 1981, the City doubled the treatment plant capacity to 3.2 million gallons per day (mgd) with the addition of a headworks, a second contact stabilization unit and a second final clarifier. With these upgrades, the treatment plant operated with redundant processes, allowing the City to take certain units off line for periodic maintenance. Around the same time, the three primary lift stations were upgraded to match the treatment plant capacity, however peak flows in excess of 3.7 mgd still bypassed treatment and were discharged, untreated, into the river or slough(s) to avoid a washout of the clarifier sludge blankets.

In 1991 substantial improvements were made to the system which included upgrades to the lift stations (standby generators, new valving, and sealing overflows), the collection system (over 15,000 lineal feet of new pipe, over 75 new & rehabilitated manholes, and over 200 service connections) and the treatment plant (new site work, electrical, structural, instrumentation and control, and safety upgrades, new fine bubble diffusers were added as well as many ancillary items which aided in the treatment process).

In 2000, various units of the treatment plant were upgraded to increase treatment capacity. A new headworks capable of handling 6.5 mgd was installed. Two parallel treatment units have a combined capacity of 4.3 mgd; however a 190,000 gallon surge tank is available to diffuse large peak flows which, by design, allows the plant to treat isolated peak flows up to 6.5 mgd.

In 2000, the Oregon Department of Environmental Quality and the City of Toledo entered into a Mutual Agreement and Order (MAO) which mandated that the City implement a de-chlorination process to reduce the amount of total residual chlorine being released into the river. The City installed a de-chlorination system in May of 2009 and has recorded a substantial reduction in the total chlorine residual.

### 2.2. Previous Planning Efforts

The following provides a summary of the relatively recent wastewater planning efforts.

1. Wastewater Facilities Plan: Completed in June 1988 by Westech Engineering recommended the above mentioned 1991 improvements.

2. Wastewater Facilities Plan: Completed in December 1993 by Clearwater Engineering Corporation, the Facilities Plan includes recommendations for improvements in the collection system and the treatment facilities.

3. Wastewater Master Plan: The City's wastewater master plan was completed in August of 1995 by Clearwater Engineering Corporation. The Plan continues the recommendations made in the 1993 Facilities Plan and recommends a schedule and funding sources for completing them.

4. Inflow and Infiltration (I/I) Study: The City commissioned Civil West to complete an inflow and infiltration study which was completed in May, 2011. A copy of the I/I study can be found in Appendix C of this report. This study is based on the following three surveys and resulted in the recommended improvement of 15 separate stretches of pipe as well as numerous manholes.

    a. *Systemwide Sanitary Sewer Smoke Testing*: The August 2009 survey identified numerous locations where deficiencies to the system and to private connections likely contributed to the significant I&I problems.

    b. *Flowmapping Survey*: The February 2010 survey identified several sections of pipeline which are subject to high levels of infiltration.

    c. *Television Survey*: The television survey was completed after the preceding two surveys identified key areas which were good candidates for further inspection. The survey catalogued 60 individual pipe segments totaling 10,200 feet of the approximately 98,800 feet of installed sewer pipe, however other sections were unable to be surveyed. The report recommends that the city pursue the additional inspection of 8 segments of pipe.

### 2.3. Need for This Report

The Facilities Plan completed by Clearwater Engineering was for the planning period between 1993 and 2015. The end of the planning period is quickly approaching and the City of Toledo wishes to have in place a new plan developed to identify and address current operational requirements as well as recommend needed upgrades to the wastewater systems.

While some of the improvements described in the 1993 plan were implemented, many were not. The most recent plant upgrade was completed in 2001 though the plant continues to have some operational issues today including pipe breaks, bypasses during storm events, and other operational challenges. A new raw sewage force main was recently installed to repair breaks in the old force main that was causing significant damage to the site in addition to spilling raw sewage.

The City's lift stations are showing their age and have experienced some major failures in recent years. While the City has worked hard to maintain these facilities, it is becoming increasing difficult to provide reliable service with this aging infrastructure.

Upon completion of the I/I study by Civil West, it became apparent that a more comprehensive study of the entire wastewater system would be appropriate at this time. Also, Oregon DEQ recommends that cities maintain a current wastewater facilities plan. Facilities plans typically cover a 20 year planning period maximum but may be shorter to stay abreast of planning needs for each system.

Considering the age of the existing Toledo Wastewater Facilities Plan and the condition and needs associated with the City's wastewater system, the time has come to complete a new wastewater facilities plan for Toledo.

# 3.0 Study Area Characteristics

Section
3

## 3.1. Study Area

The City of Toledo is located along the Yaquina River approximately seven miles inland from the central Oregon coast and is the only inland coastal community with a deep water channel. The City is situated on a bend in the river which represents, for the most part, the southern boundary of the city. The City is bounded by the Depot Slough to the west and, for the most part, the Olalla Slough to the east and lies south of Oregon State highway 20. The primary access route to Toledo is State Highway 20, which connects Highway 101, in Newport, with the City of Corvallis and ultimately the I-5 corridor in Albany. The highway is utilized by tourist and commercial traffic passing through the local area. A location map identifying the City of Toledo is presented in Figure 3.1.1

The study area for this Wastewater Facilities Plan includes all areas lying within the Urban Growth Boundary (UGB) for the City of Toledo. A Vicinity map depicting the study area for this plan is presented in Figure 3.1.2.

## 3.2. Physical Environment

### 3.2.1. Climate

The climate in the City of Toledo is classified as humid temperate. The City of Toledo generally experiences wet winters with mild temperatures and warm, dry summers. The majority of the precipitation occurs in the form of rainfall between the months of November and April. Snowfall is rare, 3-5 inches per year, and temperatures below freezing are recorded, on average, 30 times per year. The mean annual rainfall is on the order of 68 inches and the mean annual temperature is approximately 51° F. The average high temperature during the summer is 74° F and the average low temperature during the winter is approximately 37° F.

### 3.2.2. Floodplain

As briefly described in section 3.1, the City of Toledo is, with the exception of the southeast corner of the city, bounded by the Yaquina River, the Depot Slough and the Olalla Slough. Because wastewater lift stations, by their very nature, are at the lowest elevations, all of the City's lift stations are within areas defined on FEMA maps as susceptible to flooding during the 1% annual chance flood event. All lift stations, however, are designed to be above the 1% flood event elevation. The entirety of the wastewater treatment plant is outside of and above the FEMA flood zone. See figure 3.2.2 for the Flood Hazard Map.

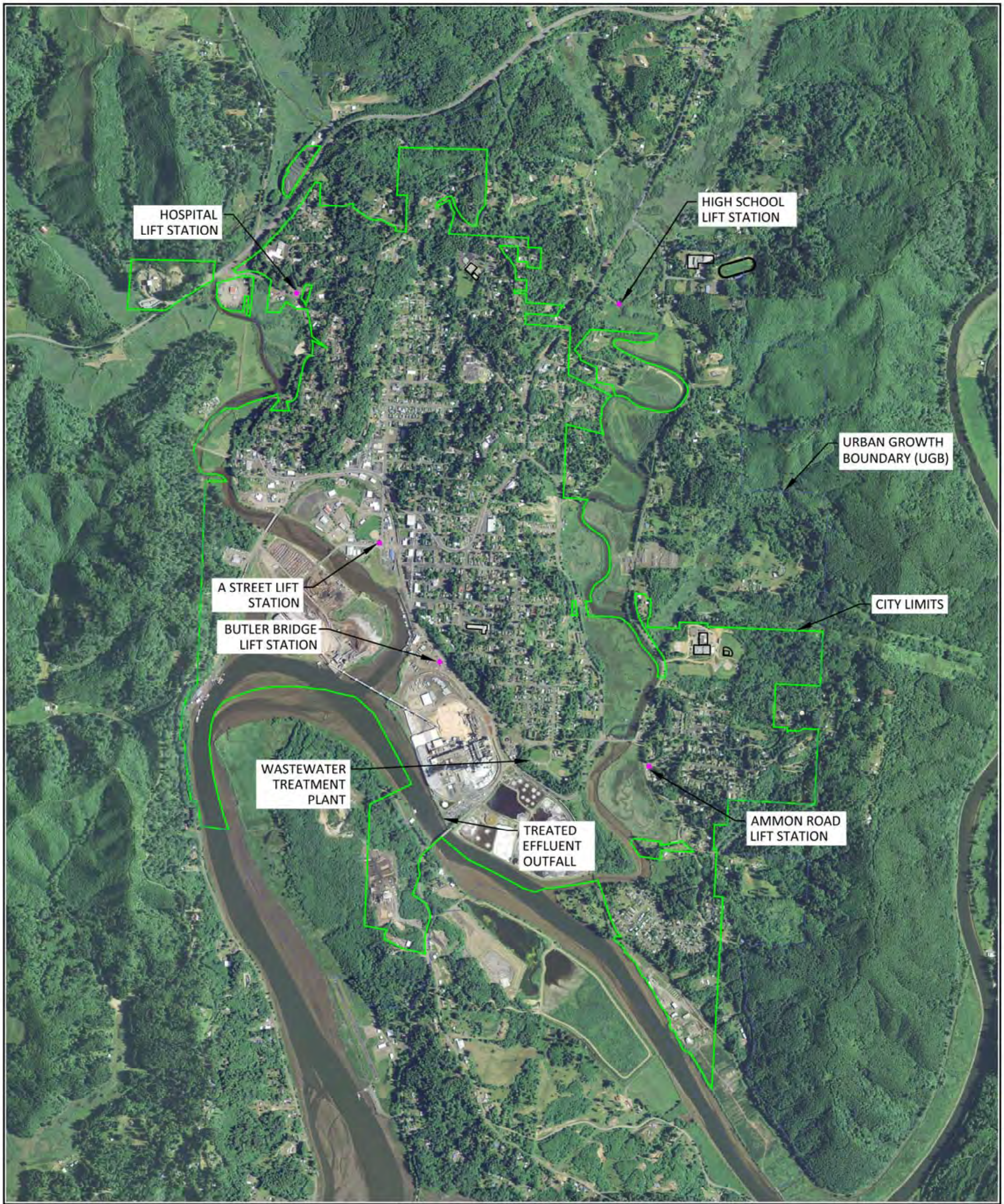
### 3.2.3. Soils

Soils within the Toledo area include a variety of sandy silt and clay. Below is a description of the various soil types found in the Toledo area:

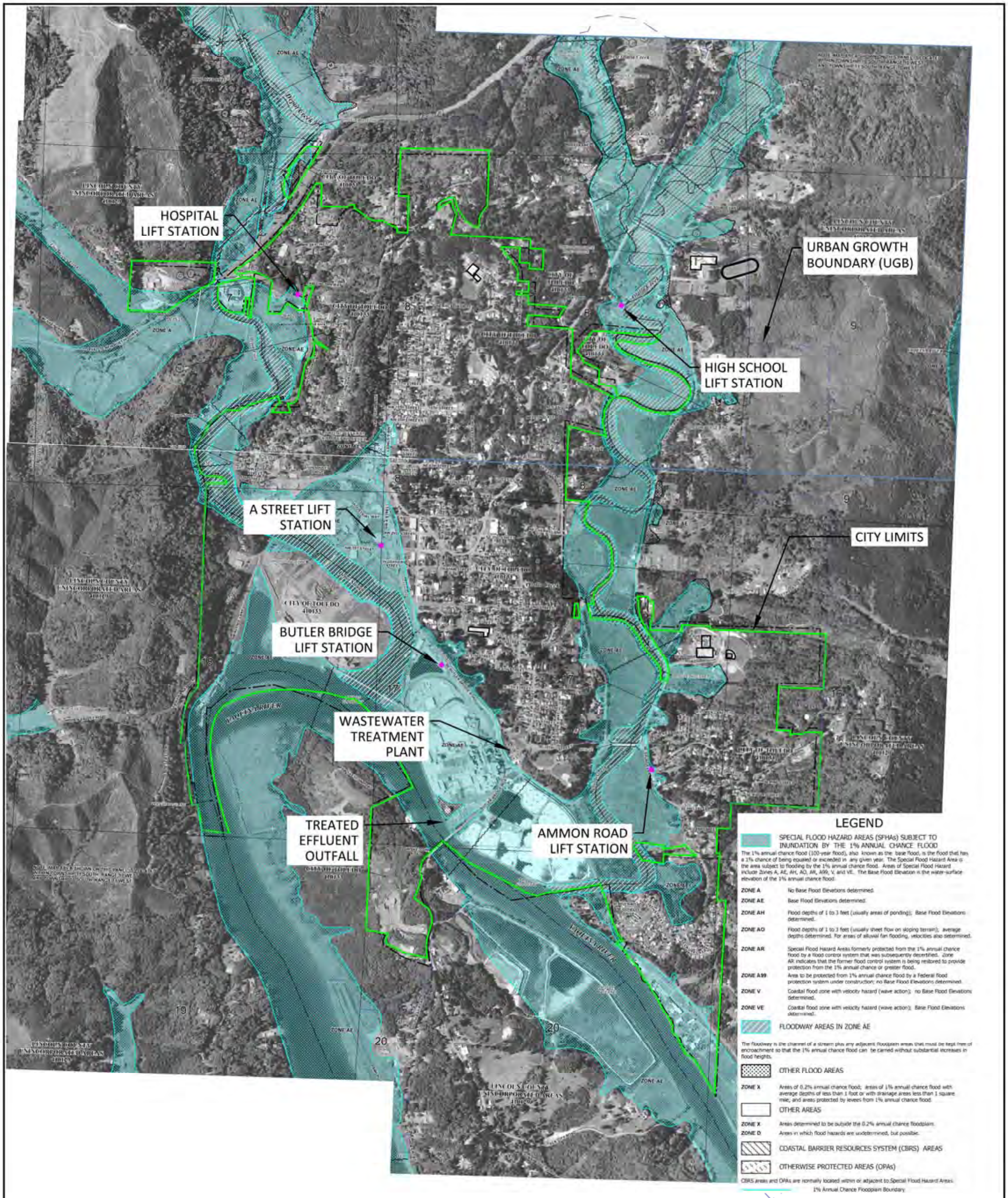
The Templeton series consists of deep, well drained soils that formed in colluvium and residuum weathered from sedimentary rocks. Templeton soils are benches, broad ridgetops, and side slopes of mountains.

The Fendall series consists of moderately deep, well drained soils formed in colluvium and residuum weathered from sedimentary rock. These soils are found on coastal hills, mountains, and old dissected marine terraces.


| Civil West Engineering Services, Inc. | Dwg By: MDW<br>Date: Dec 20, 2012 | 0 1" | **Location Map** | FIGURE |
|---|---|---|---|---|
| | Wastewater Facilities Plan | | City of Toledo<br>Lincoln County, Oregon | 3.1.1 |




| Civil West Engineering Services, Inc. | Drawn By: MLG<br>Date: Dec. 2012 | 0 1" | **Vicinity Map** | FIGURE |
|---|---|---|---|---|
| | Wastewater Facilities Plan | | City of Toledo<br>Lincoln County, OR | 3.1.2 |




Civil West Engineering Services, Inc.

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DATE: JAN. 2012

0 1"

# Flood Hazard Map

FIGURE 3.2.2

WASTEWATER FACILITIES PLAN

CITY OF TOLEDO
LINCOLN COUNTY, OR

The Knappa series consists of very deep, well drained soils that formed in alluvium derived dominantly from sedimentary rock. Knappa soils are found on coastal marine and valley terraces.

The Coquille series consists of very deep, very poorly drained soils that formed in mixed alluvium along tidal influenced flood plains.

The Brallier series consists of very poorly drained, very deep organic soils formed in partially decomposed herbaceous plant materials. Brallier soils are in depressional areas between coastal dunes and along major coastal streams.

The Bentilla series consists of deep, moderately well drained soils formed in fine textured alluvium on terraces.

The Hebo series consists of very deep, poorly drained soils that formed in alluvium of mixed materials. Hebo soils are on coastal valley and marine terraces.

The Nestucca series consists of very deep, somewhat poorly drained soils that formed in recent alluvium. Nestucca soils are typically found in flood plains.

The Brenner series consists of very deep, poorly drained soils on flood plains. They formed in recent alluvium derived from mixed sources.

### 3.3. Population Data

The 2010 population of the City of Toledo was 3465 persons, according to the 2010 Census data. In addition, the population outside the City, but within the Urban Growth Boundary (UGB) was approximately 255 persons.

Per population projections by the Oregon Office of Economic Analysis, Department of Administrative Services the growth rate for Lincoln County within the 20 year planning period will vary from 0.77% to 0.61% per year. For the purposes of this planning effort, it is assumed that the population of Toledo and its UGB will see the same growth rates.

To be conservative, it is also assumed that the portion of the population within the UGB south of the river will be connected during the planning period, increasing the flow rate into the Butler Bridge Lift Station and the treatment plant based on the per capita rate discussed in Section 5 of this report.

Table 3.3 below summarizes the anticipated growth rate in the City and UGB during the planning period covered by this plan.

**Table 3.3 - Population Projections**

| Population Projections | | | |
|---|---|---|---|
| Year | Population | | |
| | City of Toledo | Toledo UGB | Total |
| 2010 (1) | 3465 (2) | 255 | 3720 |
| 2015 | 3600 | 265 | 3865 |
| 2020 | 3718 | 274 | 3992 |
| 2025 | 3841 | 283 | 4124 |
| 2030 | 3964 | 292 | 4256 |
| 2032 (3) | 4013 | 295 | 4308 (4) |
| 2035 | 4086 | 301 | 4387 |

(1) 2010 data based on 2010 US Census
(2) Current population served by wastewater system
(3) The year 2032 represents the end of the planning period
(4) Total includes persons not currently served by the collection system but which may be connected by the end of the planning period

### *3.4. EDU Analysis*

Based on water sales records, the average quantity of water sold to a typical single family dwelling unit inside the service area is 5,350 gallons per month. This volume sold per month becomes the basis for Equivalent Dwelling Unit (EDU) calculations with 1 EDU = 5350 gallons per month of metered water sales. Since sewer fees are charged based on water usage, the same EDU definition will apply to the wastewater system as the potable water system.

Based on water sales, and excluding industrial users, the current EDU count is estimated at 1531 sewer EDUs. This number is the basis for the rate analysis in Section 9 of this report.

# 4.0 Existing Wastewater Facilities

Section
4

This chapter provides a detailed description of the existing wastewater conveyance and treatment facilities as well as an evaluation of their condition and capacity. Information presented in this chapter has been obtained from the WWTP operators and other City staff, field reconnaissance, WWTP operating records, project drawings, as-built drawings, and from the City's previous planning efforts.

The City of Toledo's Wastewater Facilities include approximately:

- 655 Manholes
- 115,638 linear feet of gravity sewer main.
- 5 lift stations
- 6000 linear feet of pressure force main
- Wastewater treatment plant
- 1500 linear feet 18" effluent discharge pipe to the Yaquina River

## *4.1. Existing Gravity Collection System*

The existing wastewater collection system includes approximately 655 manholes and 115,638 linear feet of gravity sewer main. The material and condition of the gravity main varies widely, as some of the original clay pipes installed in 1926 are still in service while other sections were installed or replaced with PVC pipe within the past few years. Reference the 2011 I/I Study in Appendix C for a comprehensive analysis of the collection system.

Some downstream sections of pipe are 10, 12, 15, and 18 inch diameter, while the majority of the system pipes are 8" diameter.

See Figures 4.1, 4.1a, 4.1b, 4.1c and 4.1d for collection system maps.

Table 4.1 below summarizes the length and size of pipe in each collection system basin.

**Table 4.1 - Basin Sewer Pipe Summary**

| Basin Sewer Pipe Summary (Feet) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Basins | Pipe Size (Inches) | | | | | | | |
| | 4 | 6 | 8 | 10 | 12 | 14 | 15 | 18 |
| A | 250 | 1,651 | | | | | | |
| B | 5,550 | 833 | 13,833 | | | | | |
| C | 2,350 | 100 | 7,932 | | | | | |
| D | 3,950 | 150 | 10,689 | | | | | |
| E | 300 | | 1,016 | | | | | |
| F | 6,950 | 718 | 8,221 | 593 | 804 | | 798 | 295 |
| G | 3,250 | 1,921 | 5,899 | | | | | |
| H | 870 | | 5,899 | | | | | |
| I | 10,450 | 996 | 13,585 | 573 | 309 | | | 1,855 |
| J | 100 | | 749 | | | | | |
| K | 4,200 | 2,120 | 5,936 | | | | | |
| L | 3,550 | | 4,292 | | 34 | | | |
| M | 2,450 | | 3,829 | 2,017 | | | | |
| N | 1,350 | 250 | 4,169 | 47 | 1,375 | | | |
| O | 4,800 | | 11,747 | | 354 | | 17 | |
| P | 5,150 | | 7,464 | | | | | |

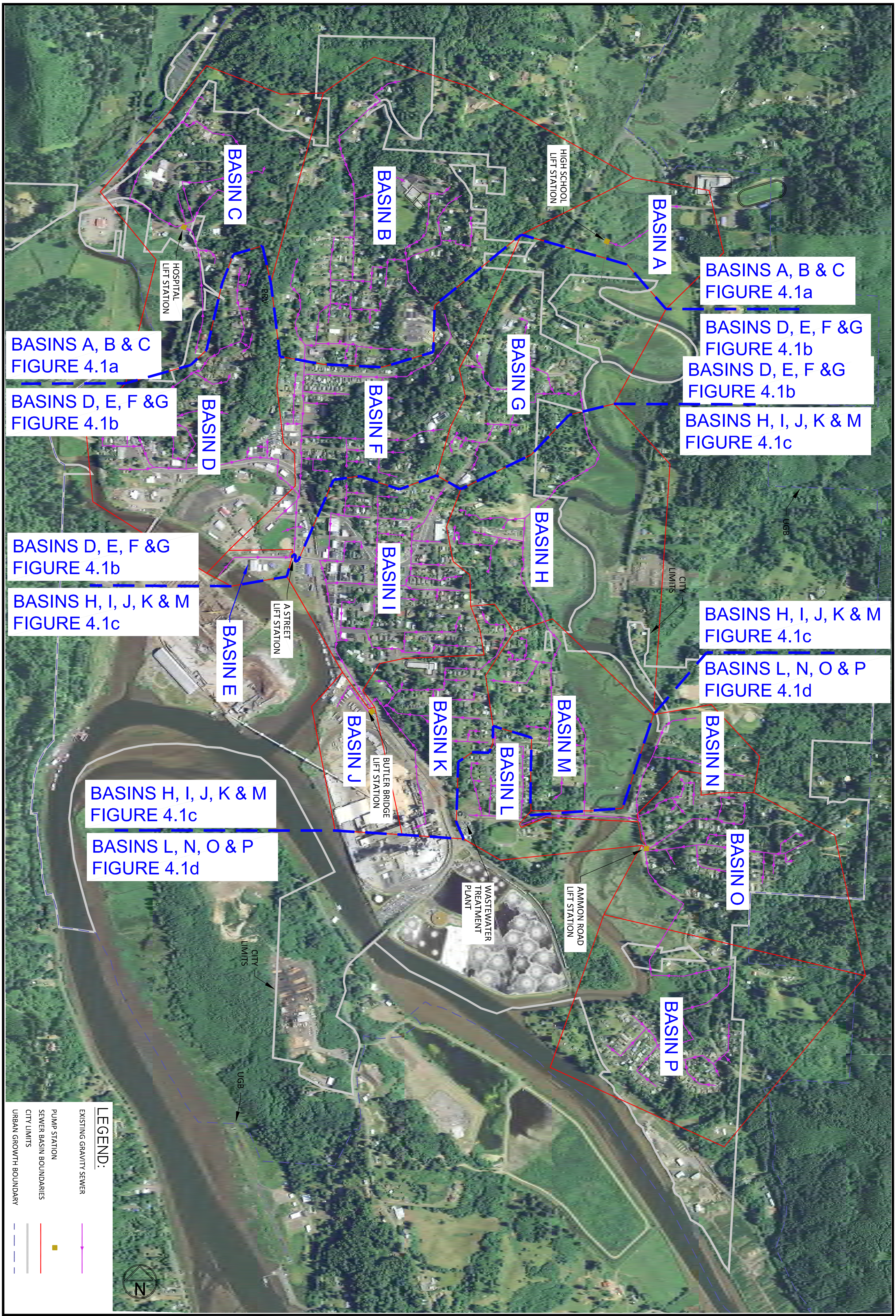
## *4.2. Existing Lift Stations and Forcemains*

There are five lift stations which are required to provide service to the residential and commercial customers within the City's Urban Growth Boundary. These include the A Street Lift Station, the Ammon Road Lift Station, the Lincoln Way Lift Station (formerly known as the Hospital Lift Station), the High School Lift Station and the Butler Bridge Lift Station.

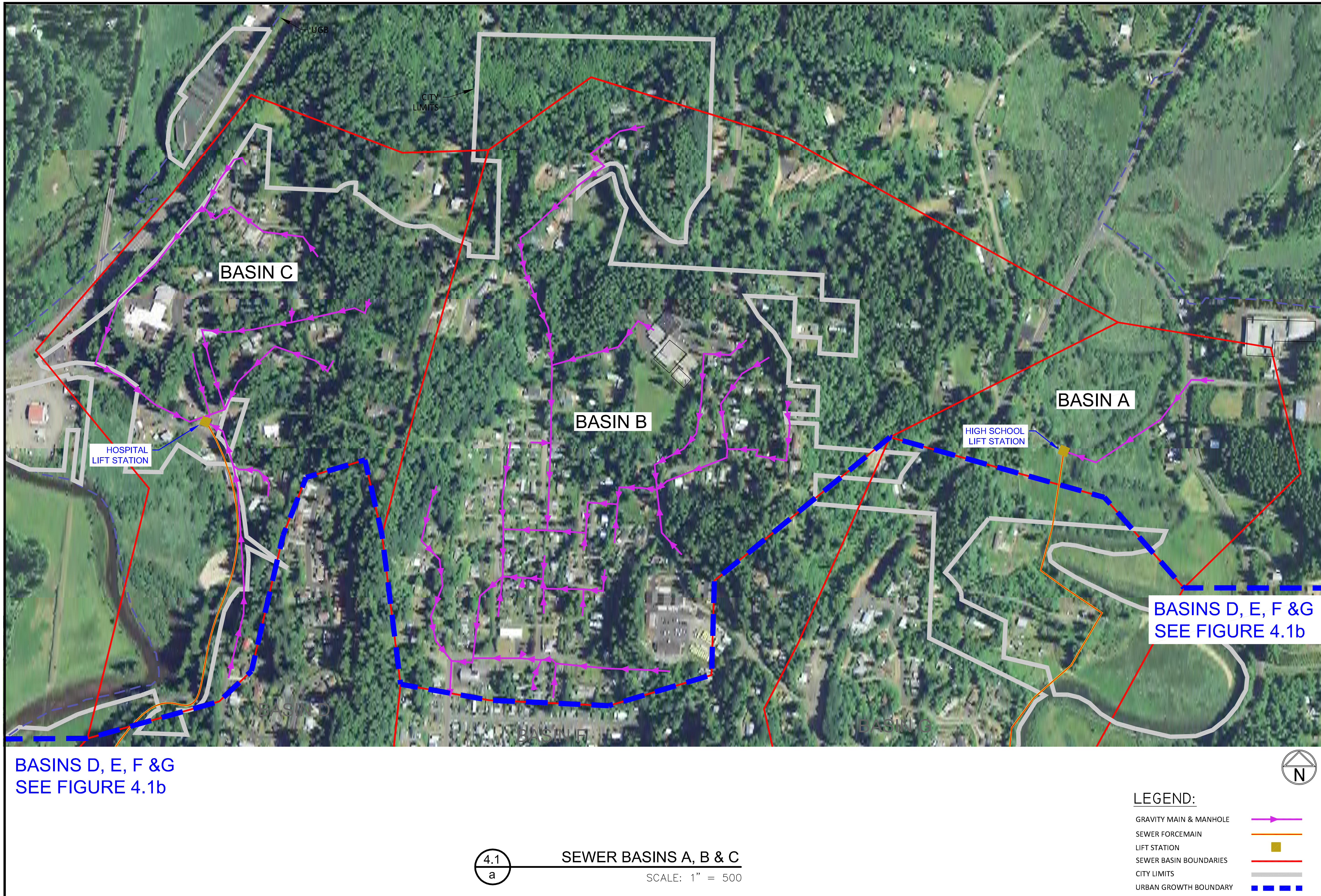
The Lincoln Way Lift Station pumps raw sewage into manhole D-33 which eventually drains to the A Street Lift Station. The A Street Lift Station pumps raw sewage into manhole I-2, which eventually drains to the Butler Bridge Lift Station. Butler Bridge Lift Station is one of two lift stations that pump directly into the headworks of the treatment plant.

The High School Lift Station pumps raw sewage into manhole G-1 which eventually drains to the Ammon Road Lift Station. The Ammon Road Lift Station is the other of the two lift stations that pump directly into the headworks of the treatment plant.

Each pump Station is designed differently and is faced with many issues. The following sections describe the individual lift stations and the deficiencies noted at each.




| FIGURE 4.1 | 0 1" <br> 1" = 1000' <br> DRAWN BY: MDW <br> DATE: JAN. 2012 | SEWER BASIN MAP <br> WASTEWATER FACILITIES PLAN | CITY OF TOLEDO <br> LINCOLN COUNTY, OREGON | Civil West Engineering Services, Inc. |
|---|---|---|---|---|




4.1 a SEWER BASINS A, B & C

SCALE: 1" = 500

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CITY OF TOLEDO
LINCOLN COUNTY, OREGON

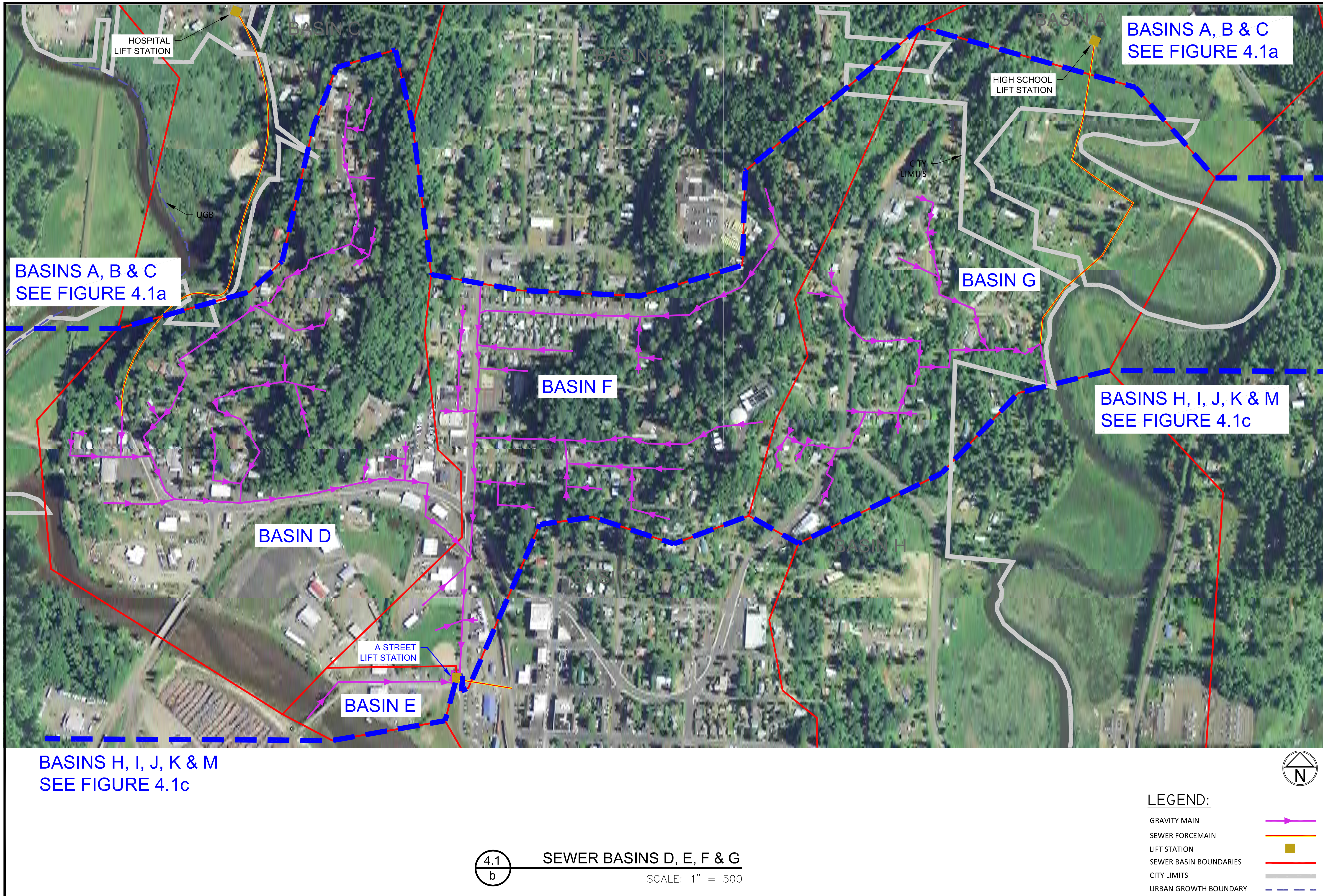
SEWER BASINS A, B & C

WASTEWATER FACILITIES PLAN

0 1"
1" = 500'

DRAWN BY: MDW
DATE: JAN. 2012

FIGURE 4.1a




4.1 b SEWER BASINS D, E, F & G

SCALE: 1" = 500

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CITY OF TOLEDO
LINCOLN COUNTY, OREGON

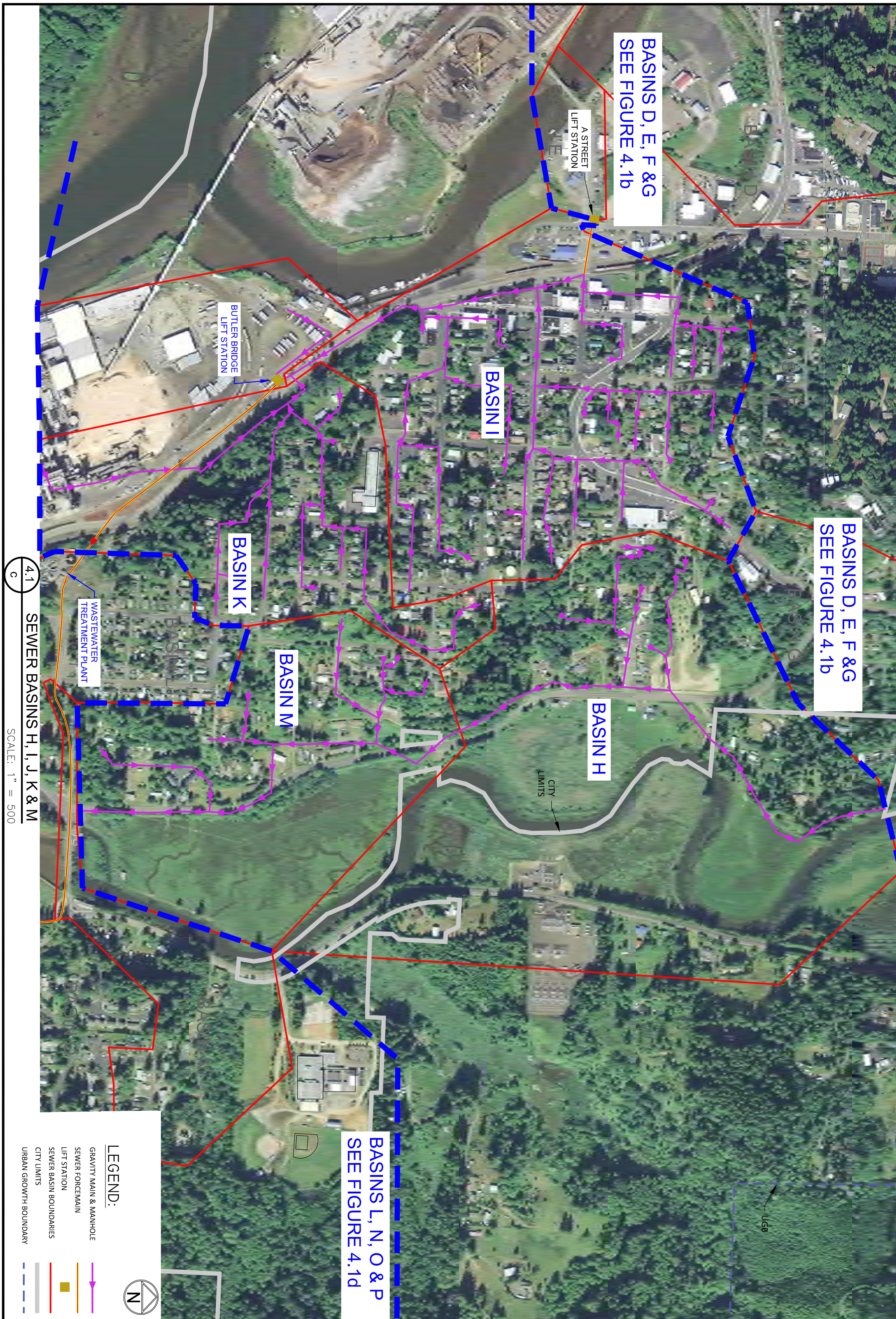
SEWER BASINS D, E, F & G

WASTEWATER FACILITIES PLAN

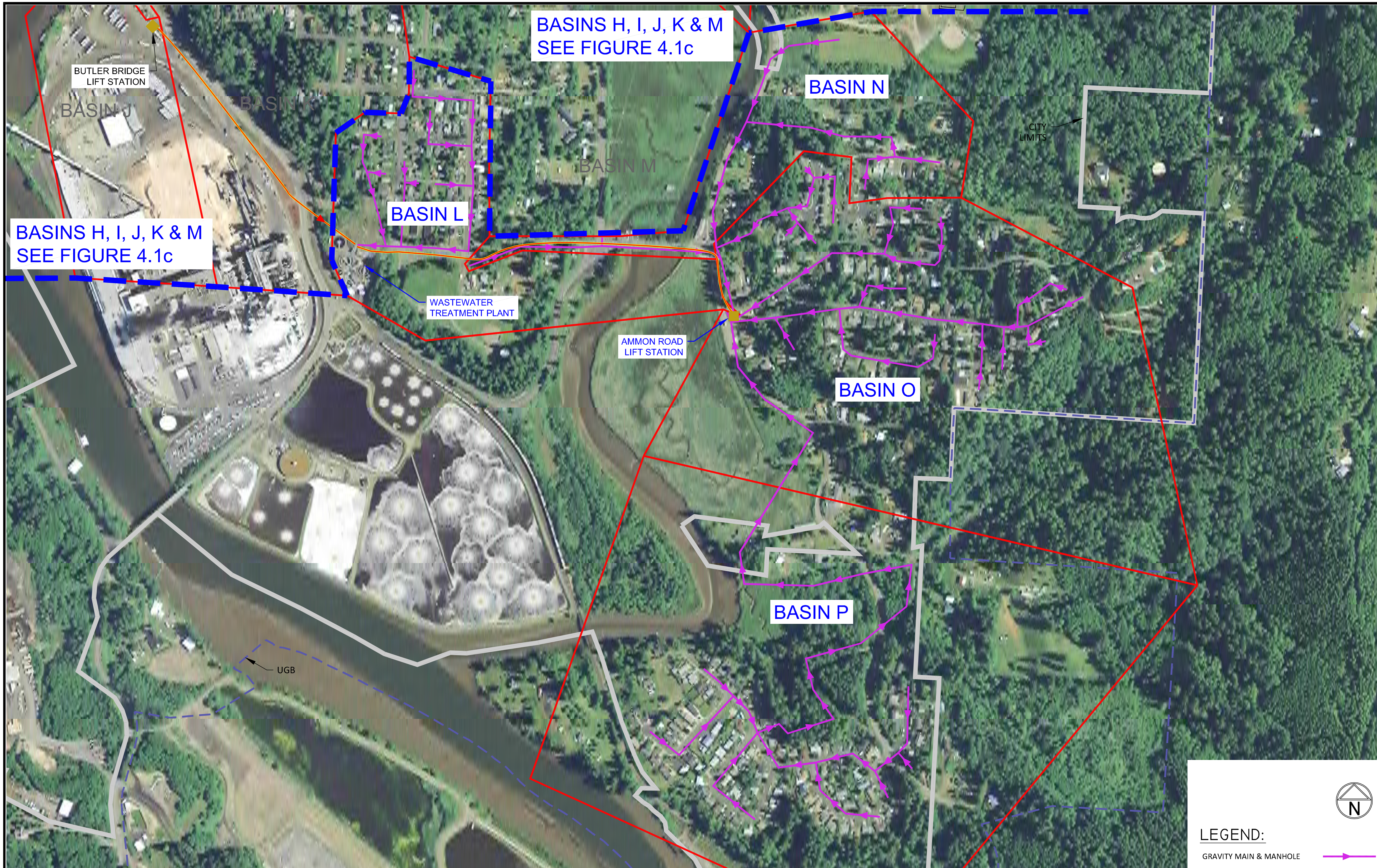
0 1"
1" = 500'

DRAWN BY: MDW
DATE: JAN. 2012

FIGURE
4.1b




| | | | | |
|---|---|---|---|---|
| FIGURE 4.1c | 0 1" <br> 1" = 500' <br> DRAWN BY: MDW <br> DATE: JAN. 2012 | SEWER BASINS H, I, J, K & M <br> WASTEWATER FACILITIES PLAN | CITY OF TOLEDO <br> LINCOLN COUNTY, OREGON | Civil West Engineering Services, Inc. |




4.1 d SEWER BASINS L, N, O & P

SCALE: 1" = 500

Civil West Engineering Services, Inc.

CITY OF TOLEDO
LINCOLN COUNTY, OREGON

SEWER BASINS L, N, O & P

WASTEWATER FACILITIES PLAN

0 1"
1" = 500'

DRAWN BY: MDW
DATE: JAN. 2012

FIGURE 4.1d

### 4.2.1. **A Street Lift Station and Forcemain**

A Street Lift Station

The A Street Lift Station is located on the northwest corner of A Street and 1st Street and serves all of Basins B, D through F, and flows from the Lincoln WayLift Station (Basin C). See Figure 4.2.1 for the A Street Lift Station Service area map. The lift station was originally constructed in 1954 and was upgraded in 1981, 1990, and 2000. The lift station has two, 20 horsepower, non-clog, centrifugal pumps, which pump the wastewater to manhole I-2 in the intersection of Butler Bridge Road and 1st Street. The design capacity of the lift station with one pump operating (firm capacity), as is normally the case, is 820 gpm (1.18 mgd), and with both pumps on is 1,250 gpm (1.75 mgd).

The pumps are set in a semicircular drywell, with the other half of the circle being the wetwell. The wetwell and drywell are over 19 feet deep, from the top of the concrete to the floor of the well. The wetwell has a volume of 853 gallons between the Lead Pump On elevation and the Lead Pump Off elevation (3.0').

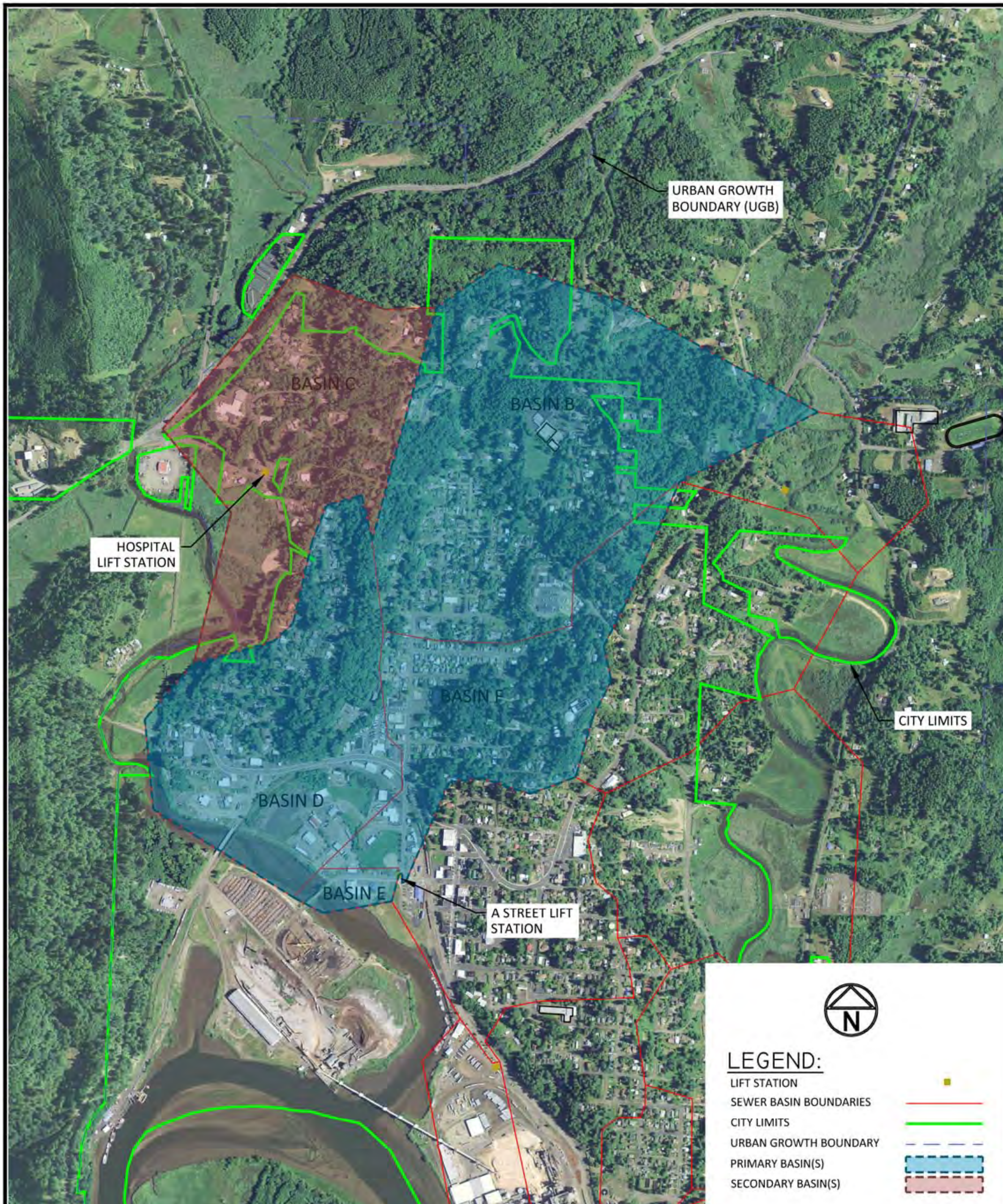
See Figures 4.2.1, 4.2.1.a, 4.2.1.b and 4.2.1.c for service basin, facility layout and schematics for the A street Lift Station

The forcemain between the A Street Lift Station and the discharge manhole is an 8" Asbestos Cement pipe which was installed with the original lift station in 1954. The forcemain is approximately 250 feet long and is continuously ascending to the discharge manhole.

Backup power at the lift station is provided by an 80 KW Diesel Generator equipped with an automatic transfer switch.

Noted deficiencies with the A Street Lift Station include:

- The lift station building is settling very badly, creating cracks in the ceiling and walls and prohibiting the doors from opening and closing correctly.
- No redundancy in the level control.
    - No operational high level float.
    - No pressure transducer.
- Dry well access is classified as a confined space under OSHA guidelines and requires notification and recording every entry into the drywell.




| | | | | |
|---|---|---|---|---|
| Civil West Engineering Services, Inc. | Drawn By: MLG<br>Date: Dec, 2012 | 0 1" | **A Street Lift Station Service Area** | FIGURE |
| | Wastewater Facilities Plan | | City of Toledo<br>Lincoln County, OR | 4.2.1 |

## DESIGN DATA

| | |
|---|---|
| Location: | 1st Street and 'A' Street |
| Type: | Duplex, Dry Pit, Flooded Suction |
| Wetwell: | Concrete Split Caisson |
| Diameter: | 12 ft |
| Area: | 38 sf |
| Volume: | 284 gal/ft depth |
| | 853 gal @ 3-ft range |
| Pump Type: | Constant Speed, Non-Clog |
| Capacity (each): | 820 gpm |
| | 1.18 MGD |
| Capacity (both): | 1250 gpm |
| | 1.80 MGD |
| Pump HP (each) | 20 HP |
| Level Control Type: | Pressure Transducer |
| Overflow Point: | Manhole F-2 |
| Level Control Type: | A Street @ Ball Park |
| Overflow Discharge: | Depot Slough |
| Average Time to Overflow | |
| ADWF: | 0.40 MGD |
| Wetwell Volume: | 853 gal @ 3-ft range |
| Influent Sewer Length: | 2000 ft |
| Inf. Sewer + MH Volume: | 5200 gal |
| Influent Sewer Invert: | -3.00 IE @ Wetwell |
| Wetwell Invert: | -11 IE Wetwell |
| Wetwell Overflow Elevation: | 8.5 EL TOS Wetwell |
| Overflow Manhole Elevation: | 7.63 Rim EL |
| Time to Overflow: | 0.30 Hours |
| Alarm Telemetry: | Autodialer |
| EPA Reliability Class: | Class I |

## CURRENT OPERATION SETTINGS

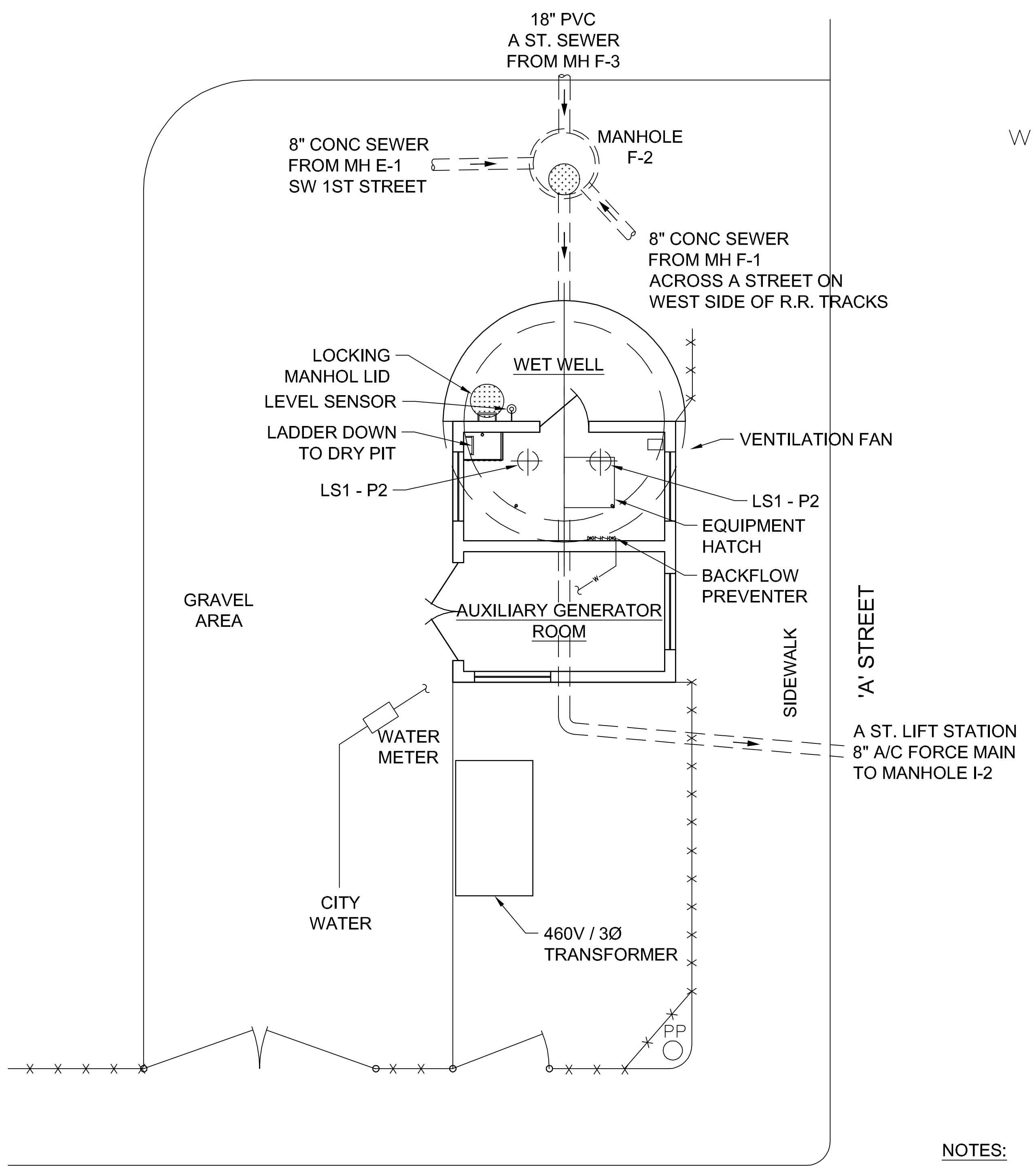
| | |
|---|---|
| Wetwell Invert Elevation | -11.0 ASL |
| Low Low Level (Alarm) | 1400.0 |
| Low Level (Alarm) | 1430.0 |
| Lead Pump Off | 1850.0 |
| Lag Pump Off | 2100.0 |
| Lead Pump On | 2100.0 |
| Lag Pump On | 2250.0 |
| High Level (Alarm) | 2550.0 |
| High High Level (Alarm) | 2800.0 |

## FORCE MAIN

| | |
|---|---|
| Pipe Material: | Asbestos Cement (1954) |
| Length: | 250 ft |
| Diameter: | 8 inch (0.35 sf) |
| Force Main Velocity | |
| (1) Pump: | 5.2 fps @ 820 gpm |
| (2) Pumps: | 8 fps @ 1250 gpm |
| Detention Time: | 5 minutes |
| Volume - Force Main: | 653 gallons |
| Volume - Wetwell: | 853 gallons |
| Volume - FM+WW: | 1505 gallons |
| ADWF: | 278 gpm |
| FM Detention Time: | 48 sec @ 820 gpm |
| | 31 sec @ 1250 gpm |
| Profile: | Continuously Ascending, |
| Discharge MH: | Manhole I-2 |
| Air Release Valves: | None |
| Vacuum Release Valves: | None |

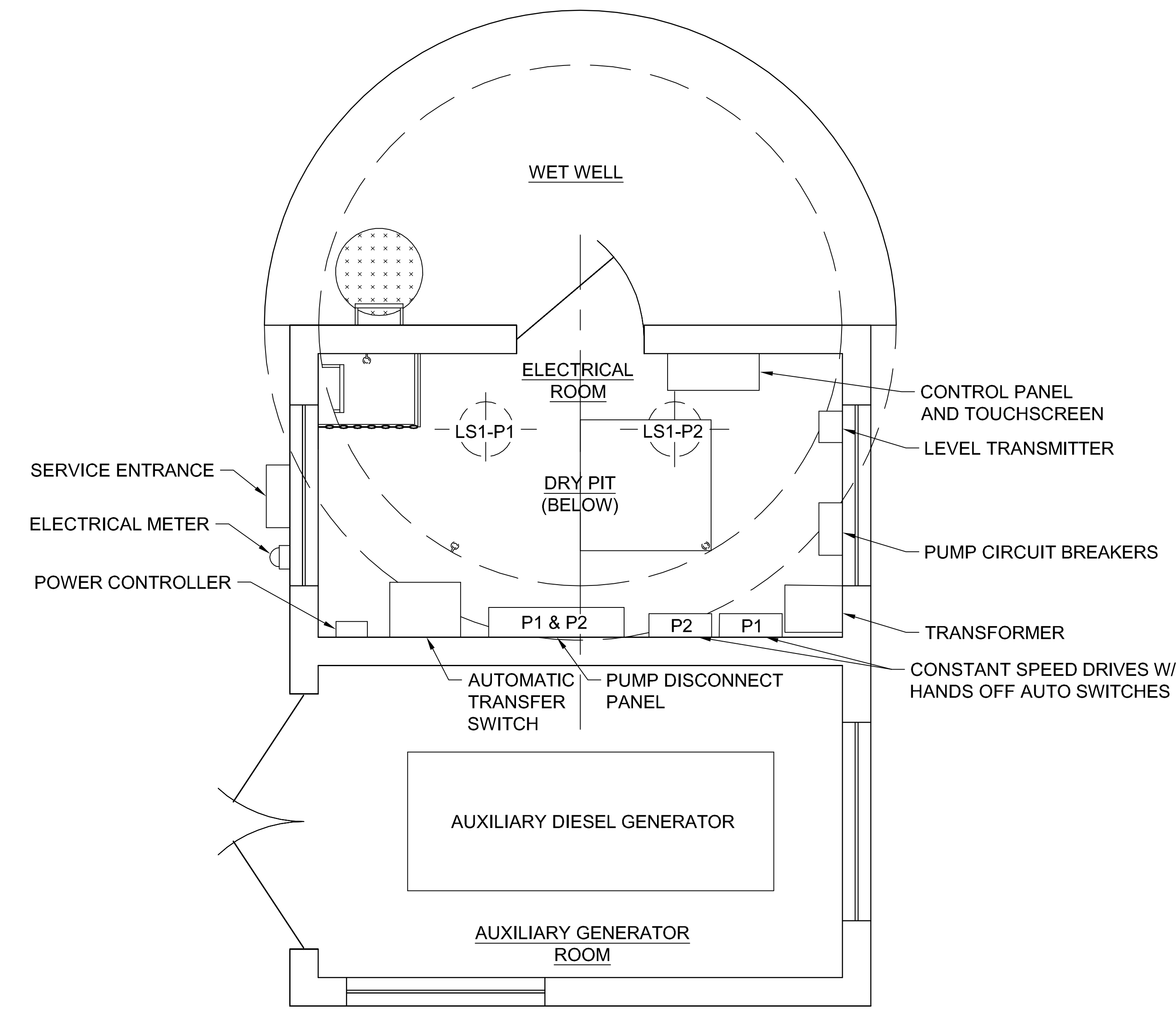
## AUXILIARY POWER

| | |
|---|---|
| Type: | Diesel Generator |
| Location: | On-Site |
| Output: | 80 KW |
| Fuel Tank Capacity: | 50 gallons |
| Transfer Switch: | Automatic |




## A STREET LIFT STATION SITE LAYOUT

1" = 10'




## ELECTRICAL AND GENERATOR PLAN

1" = 4'

NOTES:

1. A STREET LIFT STATION WAS CONSTRUCTED IN 1955 AND EQUIPPED WITH NEW ELECTRICAL EQUIPMENT IN 2000.
2. SEE AS-BUILT DRAWINGS ENTITLED TOLEDO SCHEDULE A PUMP STATION IMPROVEMENTS *(CLEARWATER, 2000)*; SANITARY SEWER SYSTEM REHABILITATION *(WESTECH, 1990)*; SEWAGE LIFT STATION AND FORCE MAIN IMPROVEMENTS *(WESTECH, 1981)*; AND CONTRACT DOCUMENTS FOR THE CONSTRUCTION OF SEWAGE TREATMENT PLANT AND PUMPING STATIONS AND AN INTERCEPTOR SEWER *(CH2M, 1954)*.

Civil West Engineering Services, Inc.

CITY OF TOLEDO
LINCOLN COUNTY, OREGON

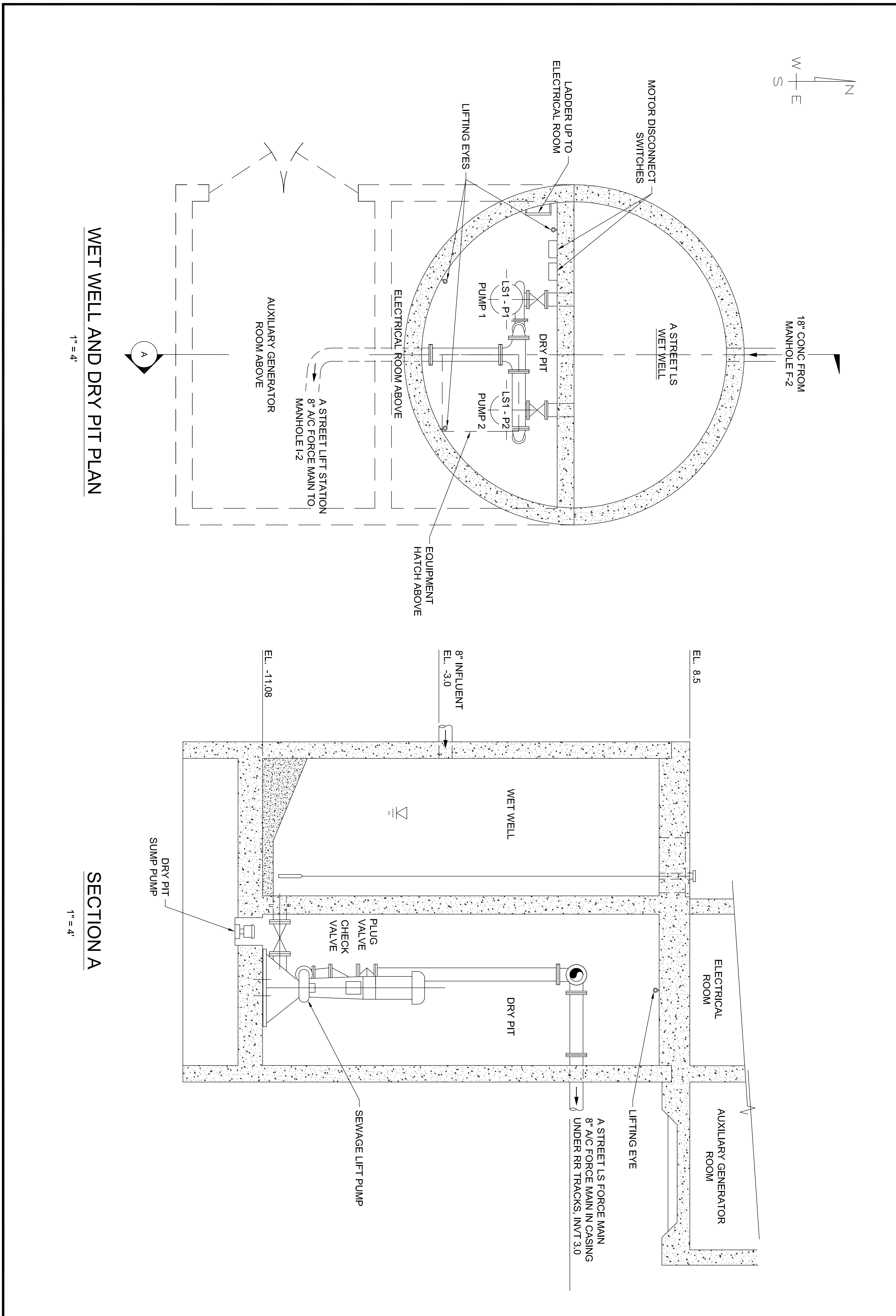
**A STREET LIFT STATION SITE PLAN**

WASTEWATER FACILITIES PLAN

0 1"
SCALE VARIES

DRAWN BY: MDW
DATE: NOV. 22, 2011

FIGURE
4.2.1b




| | | | | |
|---|---|---|---|---|
| FIGURE 4.2.1c | 0 — 1"<br>1" = 4'<br>Drawn By: MDW<br>Date: Nov. 22, 2011 | **A STREET LIFT STATION MECHANICAL PLAN**<br>Wastewater Facilities Plan | City of Toledo<br>Lincoln County, Oregon | Civil West Engineering Services, Inc. |

### 4.2.2. **Ammon Road Lift Station**


Ammon Road Lift Station

The Ammon Road Lift Station is located on the southeast corner of Ammon and Sturdevant Roads and serves all of Basins G, H and L through O, and flows from the High School Lift Station (Basin A). See Figure 4.2.2 for the Ammon Road Lift Station Service area map. The lift station was originally constructed in 1954 and was upgraded in 1983, 1990, and 2000. The lift station has two, 50 horsepower, non-clog, centrifugal pumps, which pump the wastewater to the headworks of the treatment plant. The design capacity of the lift station with one pump operating, as is normally the case, is 820 gpm (1.18 mgd), and with both pumps on is 1,390 gpm (2.0 mgd).

The pumps are set in a semicircular drywell, with the other half of the circle being the wetwell. The wetwell and drywell are over 15 feet deep, from the top of the concrete to the floor of the well. The wetwell has a volume of 853 gallons between the Lead Pump On elevation and the Lead Pump Off elevation (1.0').

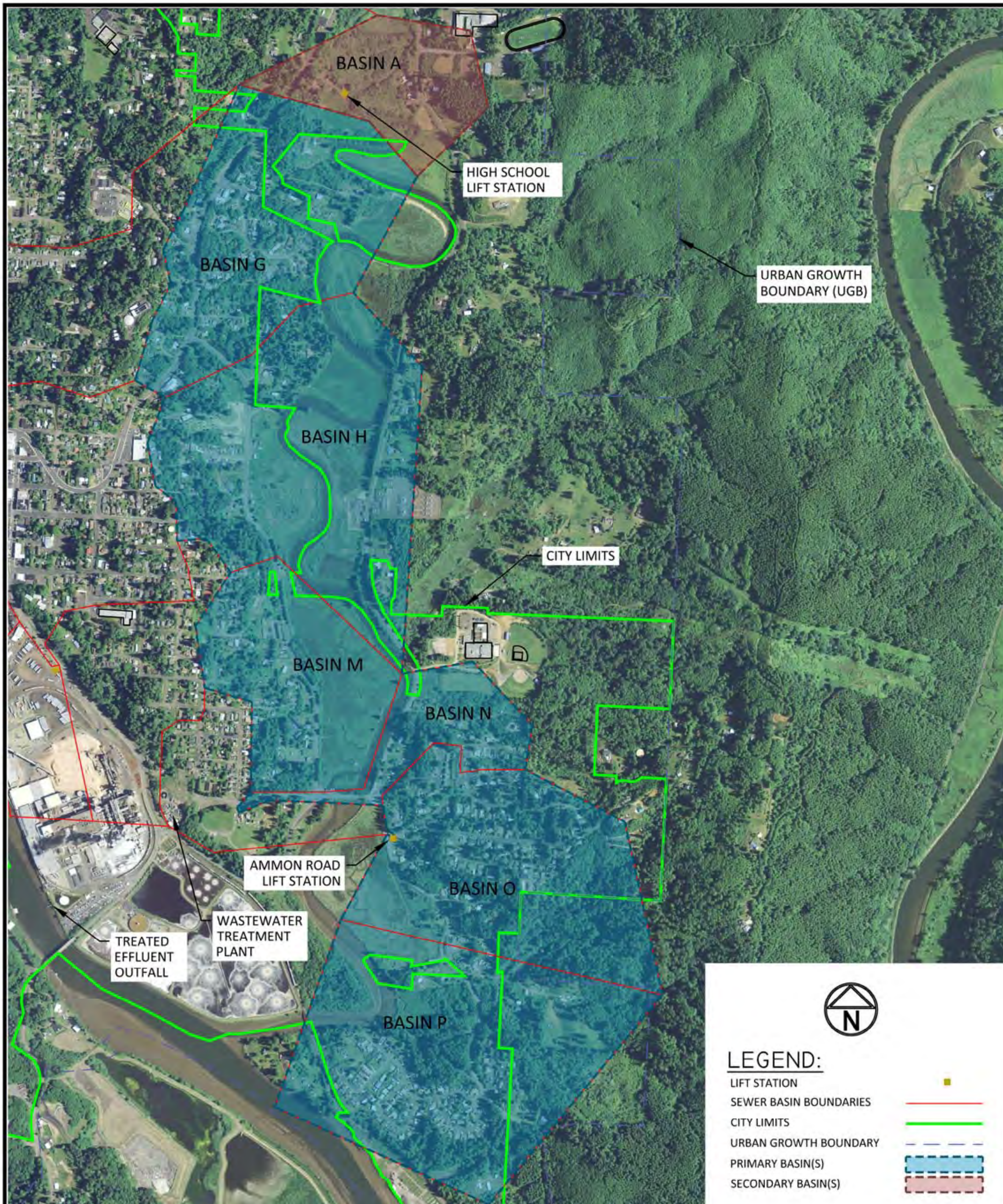
See Figures 4.2.2, 4.2.2.a, 4.2.2.b and 4.2.2.c for service basin, facility layout and schematics for the Ammon Road Lift Station.

The forcemain between the Ammon Road Lift Station and the discharge at the treatment plant is a 10" pipe which was installed in 1999/2000. The forcemain is approximately 2520 feet long and has variable slopes throughout its length. It has one Air/Vacuum Release Valve at the high point in the line near 10th Street.

Backup power at the lift station is provided by an 80 KW Diesel Generator equipped with an automatic transfer switch.

Noted deficiencies with the Ammon Road Lift Station include:

- The lift station building is settling creating cracks in the ceiling and walls and prohibiting the doors from opening and closing correctly.
- No redundancy in the level controls.
- Dry well access is classified as a confined space under OSHA guidelines and requires notification and recording every entry into the drywell.
- No ability to bypass pump at the lift station.
- The partition wall separating the wet and dry wells is leaking.
- Electrical within the pit needs to be updated, no explosion proof lighting in pit.




| Civil West Engineering Services, Inc. | Drawn By: MLG<br>Date: Dec, 2012 | 0 1" | Ammon Road Lift Station Service Area | FIGURE |
|---|---|---|---|---|
| | Wastewater Facilities Plan | | City of Toledo<br>Lincoln County, OR | 4.2.2 |

## DESIGN DATA

| | |
|---|---|
| Location: | Sturdevant Road, between Ammon Road and Alder Lane |
| Type: | Duplex, Dry Pit, Flooded Suction |
| Wetwell: | Concrete Split Caisson |
| Diameter: | 12 ft |
| Area: | 38 sf |
| Volume: | 284 gal/ft depth |
| | 853 gal @ 3-ft range |
| Pump Type: | Variable Speed, Non-Clog |
| Capacity (each): | 820 gpm |
| | 1.18 MGD |
| Capacity (both): | 1,390 gpm |
| | 2.00 MGD |
| Pump HP (each) | 50 HP |
| Level Control Type: | Pressure Transducer |
| Overflow Point: | Manhole N-5 |
| Level Control Type: | 10th Street & East Slope Road |
| Overflow Discharge: | Olalla Slough |
| Average Time to Overflow | |
| ADWF: | 0.16 MGD |
| Wetwell Volume: | 853 gal @ 3-ft range |
| Influent Sewer Length: | 4,200 ft |
| Inf. Sewer + MH Volume: | 18,200 gal |
| Influent Sewer Invert: | 5.00 IE @ Wetwell |
| Wetwell Invert: | -5.30 IE Wetwell |
| Wetwell Overflow Elevation: | 9.10 EL TOS Wetwell |
| Overflow Manhole Elevation: | 8.96 Rim EL |
| Time to Overflow: | 3.00 Hours |
| Alarm Telemetry: | Autodialer |
| EPA Reliability Class: | Class I |

## CURRENT OPERATION SETTINGS

| | | |
|---|---|---|
| Wetwell Invert Elevation | -11.0 ASL | |
| Low Low Level (Alarm) | 705.0 | -8.520486758 |
| Low Level (Alarm) | 730.0 | -8.432560757 |
| Lead Pump Off | 855.0 | -7.992930749 |
| Lag Pump Off | 855.0 | -7.992930749 |
| Lead Pump On (Min Speed) | 1280.0 | -6.498188724 |
| Lead Pump On (Max Speed) | 1330.0 | -6.322336721 |
| Lag Pump On (Min Speed) | 1380.0 | -6.146484718 |
| Lag Pump On (Max Speed) | 1430.0 | -5.970632716 |
| High Level (Alarm) | 1480.0 | -5.794780713 |
| High High Level (Alarm) | 1530.0 | -5.61892871 |

Civil West Engineering Services, Inc.

DRAWN BY: MDW
DATE: NOV. 29, 2011

0 1"

WASTEWATER FACILITIES PLAN

AMMON ROAD LIFT STATION DESIGN DATA

CITY OF TOLEDO
LINCOLN COUNTY, OR

FIGURE 4.2.2a

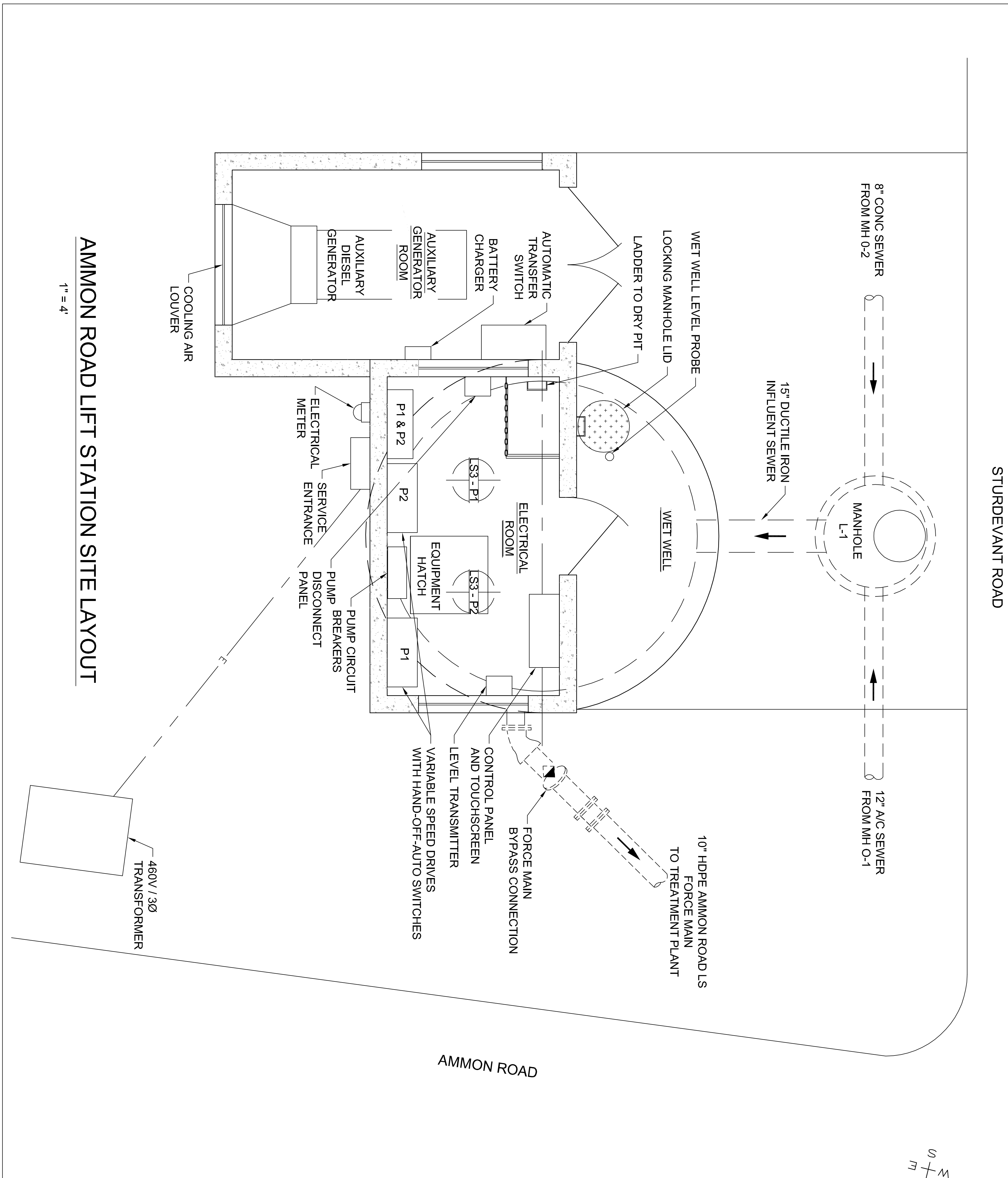



FIGURE 4.2.2b

0 1"
1" = 4'

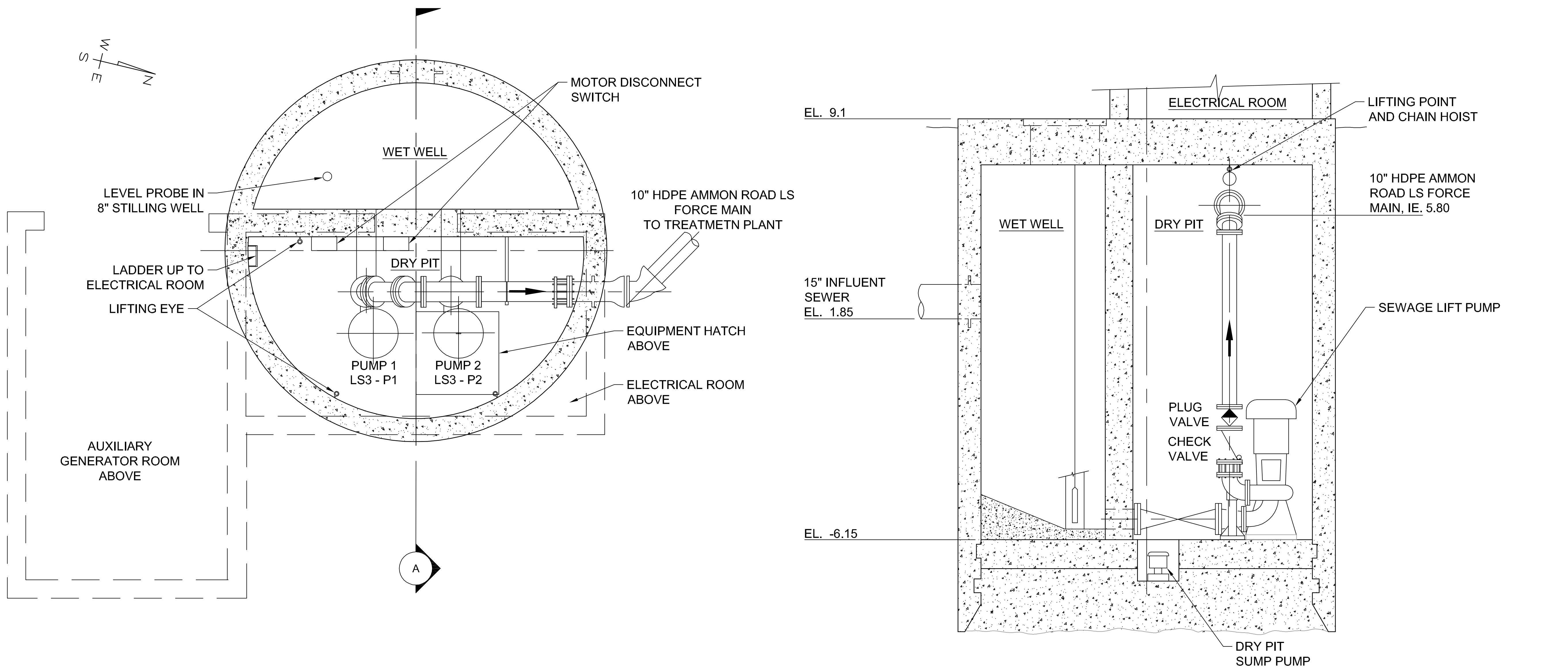
DRAWN BY: MDW
DATE: NOV. 22, 2011

**AMMON ROAD LIFT STATION SITE PLAN**

WASTEWATER FACILITIES PLAN

CITY OF TOLEDO
LINCOLN COUNTY, OREGON

Civil West Engineering Services, Inc.




## WET WELL AND DRY PIT PLAN

1" = 4'

## SECTION A

1" = 4'

Civil West Engineering Services, Inc.

CITY OF TOLEDO
LINCOLN COUNTY, OREGON

**AMMON ROAD LIFT STATION MECHANICAL PLAN**

WASTEWATER FACILITIES PLAN

0 1"
1" = 4'

DRAWN BY: MDW
DATE: NOV. 22, 2011

FIGURE
4.2.2c

### 4.2.3. High School Lift Station

High School Lift Station

The High School Lift Station is located approximately 400 feet east of the intersection of Old Hwy 20 and Mossy Loop Road and serves Basin A, which is primarily the high school. See Figure 4.2.3 for the High School Lift Station Service area map. The lift station was originally constructed in 1975 and was upgraded in 2000. The lift station has two, 23 horsepower, non-clog, submersible pumps, which pump the wastewater to manhole G-1 which is approximately 400 feet southeast from the end of NE Canyon Drive. The design capacity of the lift station with one pump operating, as is normally the case, is 325 gpm (0.47 mgd), and with both pumps on is 427 gpm (0.61 mgd).

Two submersible pumps emerged in a 6' diameter wetwell. The wetwell is over 24 feet deep, from the top of the concrete to the floor of the well. The wetwell has a volume of 634 gallons between the Lead Pump On elevation and the Lead Pump Off elevation (3.0').

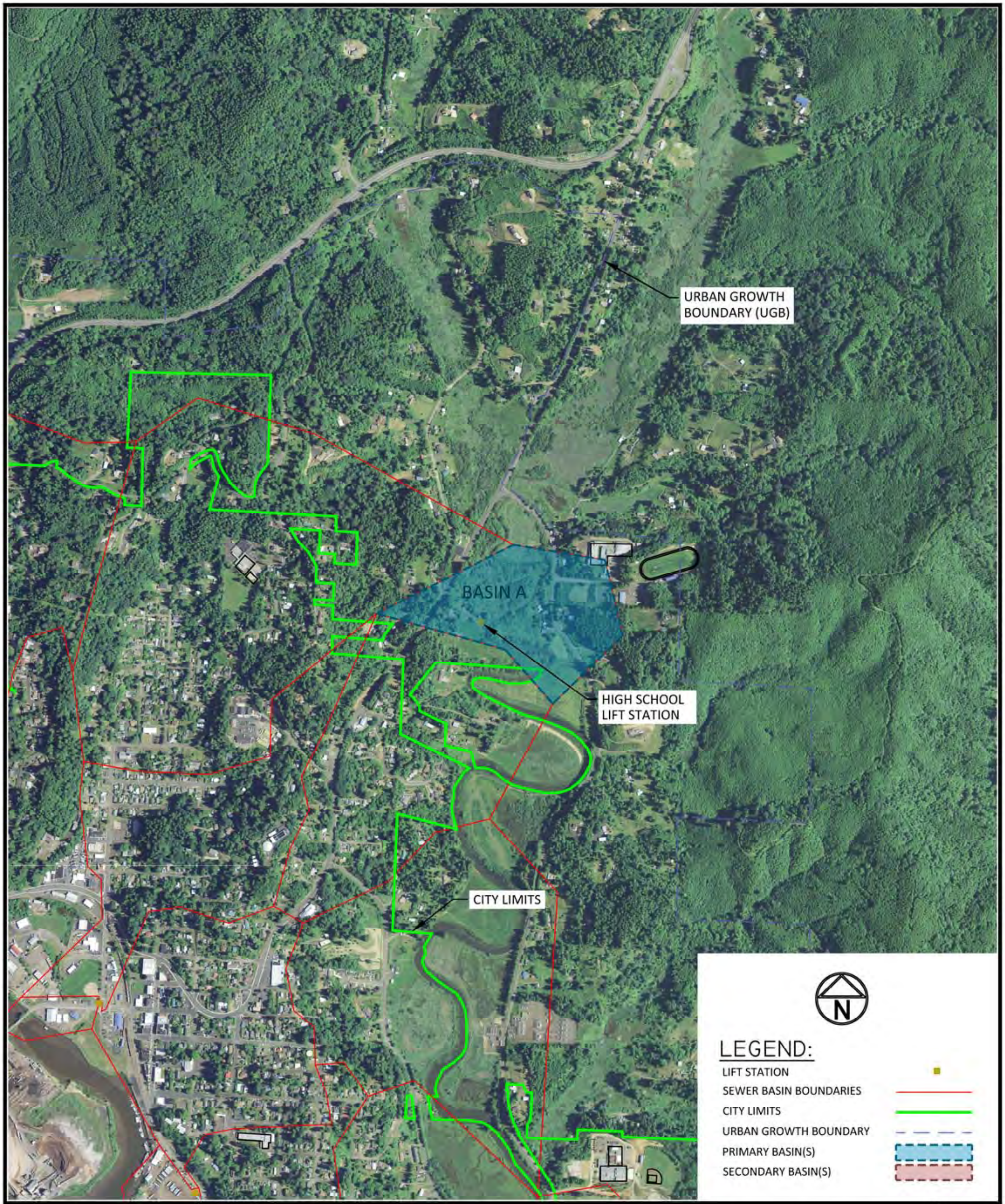
See Figures 4.2.3, 4.2.3.a, 4.2.3.b and 4.2.3.c for service basin, facility layout, schematics and design data for the High School Lift Station.

At the time of this report, the High School Lift Station does not have a dedicated, permanent backup generator, however the City is planning on moving a 94KW generator to the site for permanent backup power from a rebuild water lift station.

The forcemain between the High School Lift Station and the discharge manhole is a 6" Asbestos Cement pipe which was installed with the lift station in 1975. The forcemain is approximately 2100 feet long and has variable slopes throughout its length. It has two Automatic Combination AVRV Assemblies. The forcemain traverses unimproved properties and, as such, along the forcemain are 7 manholes located at alignment changes.

Noted deficiencies with the High School Lift Station include:

- Access door to the facility needs to be replaced.
- Facility's pressure transducer not functioning properly, new level controls may be required.
- No ability to bypass pump at the lift station.
- No ability to monitor pump station flows (no flow meter, although there are pump run-time indicators).
- No dedicated on site backup power supply, facility uses portable generator stored at WWTP.
- Groundwater leaks into the wetwell.
- Very low flows and long detention times.




Civil West Engineering Services, Inc.

Drawn By: MLG
Date: Dec, 2012

Wastewater Facilities Plan

**High School Lift Station Service Area**

City of Toledo
Lincoln County, OR

FIGURE 4.2.3

# HIGH SCHOOL LIFT STATION

**DESIGN DATA**

| | |
|---|---|
| Location: | End of private drive off of Service Road |
| Type: | Duplex Submersible |
| Wetwell: | Precast Concrete |
| Diameter: | 6 ft |
| Area: | 28 sf |
| Volume: | 211 gal/ft depth |
| | 634 gal @ 3-ft range |
| Pump Type: | Constant Speed, Non-Clog |
| Capacity (each): | 325 gpm |
| | 0.47 MGD |
| Capacity (both): | 427 gpm |
| | 0.61 MGD |
| Pump HP (each) | 23 HP |
| Level Control Type: | Pressure Transducer |
| Overflow Point: | Wetwell |
| Level Control Type: | Wetwell |
| Overflow Discharge: | Olalla Slough |
| Average Time to Overflow | |
| ADWF: | 0.05 MGD |
| Wetwell Volume: | 634 gal @ 3-ft range |
| Influent Sewer Length: | 0 ft |
| Inf. Sewer + MH Volume: | 634 gal |
| Influent Sewer Invert: | -9.80 IE @ Wetwell |
| Wetwell Invert: | -15.00 IE Wetwell |
| Wetwell Overflow Elevation: | 7.00 EL TOS Wetwell |
| Overflow Manhole Elevation: | 8.21 Rim EL |
| Time to Overflow: | 1.44 Hours |
| Alarm Telemetry: | Autodialer |
| EPA Reliability Class: | Class I |

**CURRENT OPERATION SETTINGS**

| | |
|---|---|
| Wetwell Invert Elevation | -17.0 |
| Low Low Level (Alarm) | -15.0 |
| Low Level (Alarm) | -14.0 |
| Lead Pump Off | -13.0 |
| Lag Pump Off | -12.0 |
| Lead Pump On | -10.0 |
| Lag Pump On | -9.0 |
| High Level (Alarm) | -5.0 |
| High High Level (Alarm) | 0.0 |

**FORCE MAIN**

| | |
|---|---|
| Pipe Material: | Asbestos Cement (1954) |
| Length: | 2100 ft |
| Diameter: | 6 inch (0.20 sf) |
| Force Main Velocity | |
| (1) Pump: | 3.7 fps @ 325 gpm |
| (2) Pumps: | 4.8 fps @ 427 gpm |
| Detention Time: | 107 minutes |
| Volume - Force Main: | 3084 gallons |
| Volume - Wetwell: | 634 gallons |
| Volume - FM+WW: | 3718 gallons |
| ADWF: | 35 gpm |
| FM Detention Time: | 97 min @ |
| Profile: | Varies greatly, positive and |
| Discharge MH: | Manhole G-1 |
| Air/Vac. Release Valves: | Automatic Combination |

**AUXILIARY POWER**

| | |
|---|---|
| Type: | Portable |
| Location: | WWTP |
| Output: | 65 KW |
| Fuel Tank Capacity: | 50 gallons |
| Transfer Switch: | Manual |

Civil West Engineering Services, Inc.

DRAWN BY: MDW
DATE: NOV. 29, 2011

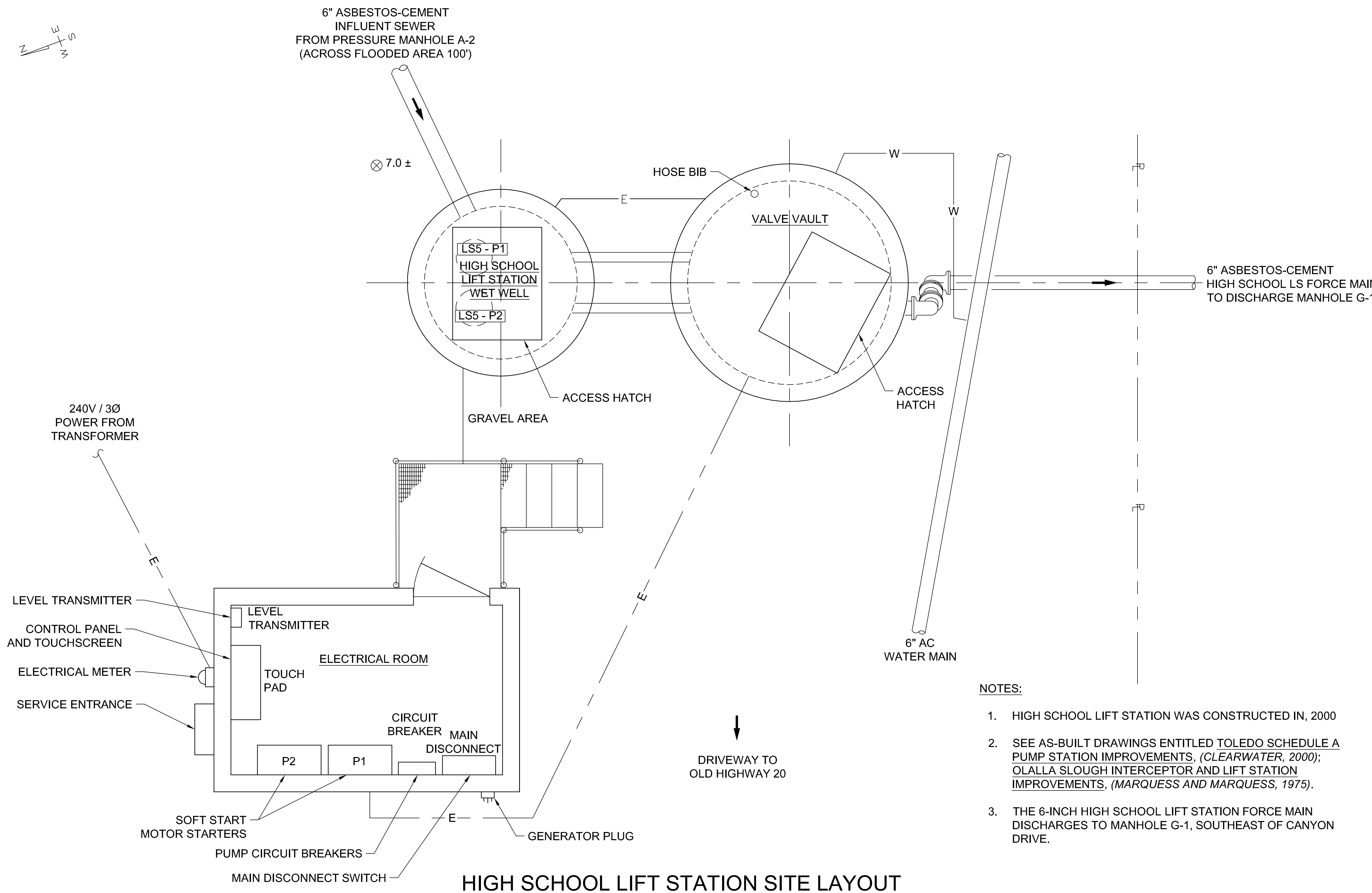
0 1"

WASTEWATER FACILITIES PLAN

**HIGH SCHOOL LIFT STATION DESIGN DATA**

CITY OF TOLEDO
LINCOLN COUNTY, OR

FIGURE 4.2.3a




HIGH SCHOOL LIFT STATION SITE LAYOUT

1" = 4'

NOTES:

1. HIGH SCHOOL LIFT STATION WAS CONSTRUCTED IN, 2000
2. SEE AS-BUILT DRAWINGS ENTITLED TOLEDO SCHEDULE A PUMP STATION IMPROVEMENTS, *(CLEARWATER, 2000)*; OLALLA SLOUGH INTERCEPTOR AND LIFT STATION IMPROVEMENTS, *(MARQUESS AND MARQUESS, 1975)*.
3. THE 6-INCH HIGH SCHOOL LIFT STATION FORCE MAIN DISCHARGES TO MANHOLE G-1, SOUTHEAST OF CANYON DRIVE.

Civil West Engineering Services, Inc.

CITY OF TOLEDO
LINCOLN COUNTY, OREGON

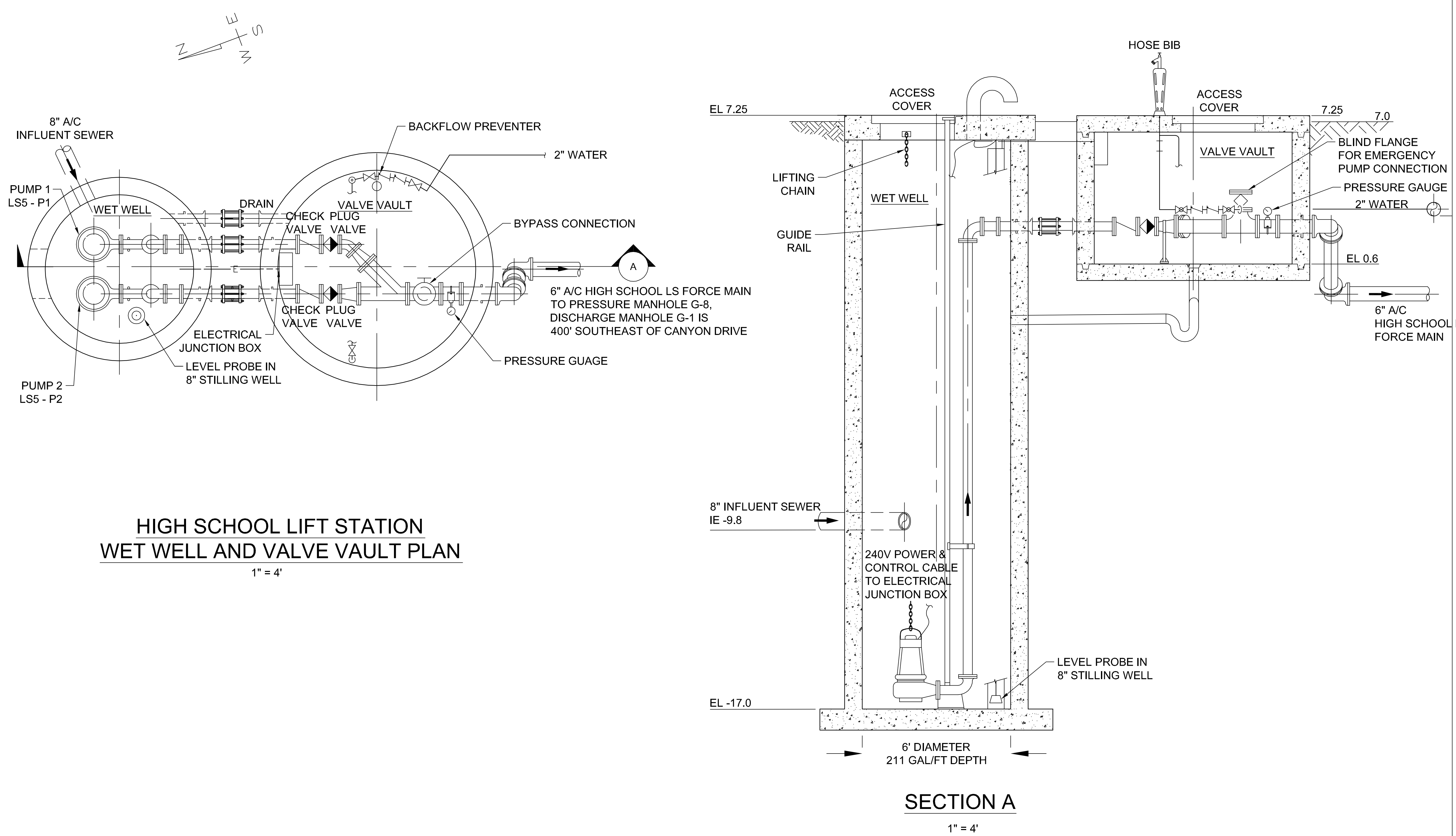
**HIGH SCHOOL LIFT STATION SITE PLAN**

WASTEWATER FACILITIES PLAN

0 1"
1" = 4'

DRAWN BY: MDW
DATE: NOV. 22, 2011

FIGURE 4.2.3b




**Civil West**
Engineering Services, Inc.

City of Toledo
Lincoln County, Oregon

**HIGH SCHOOL LIFT STATION SITE PLAN**

Wastewater Facilities Plan

0 — 1"
1" = 4'

Drawn By: MDW
Date: Nov. 22, 2011

FIGURE
4.2.3c

### 4.2.4. Lincoln Way Lift Station

Lincoln Way Lift Station

The Lincoln Way Lift Station is located on the northwest corner of Lincoln Way and Toledo Frontage Road (Hwy 20) and serves Basin C. See Figure 4.2.4 for the Lincoln Way Lift Station Service area map. The lift station was completely rebuilt in 2000. The lift station has two, 30 horsepower, non-clog, submersible pumps, which pump the wastewater to manhole D-33 which is near the intersection of the Toledo Frontage Road and NW "I" Street. The design capacity of the lift station with one pump operating, as is normally the case, is 325 gpm (0.45 mgd), and with both pumps on is 427 gpm (0.60 mgd).

The pumps are set in a 6' diameter wetwell. The wetwell is approximately 24.25 feet deep, from the top of the wetwell to the floor of the well. The wetwell has a volume of 634 gallons between the Lead Pump On elevation and the Lead Pump Off elevation (3').

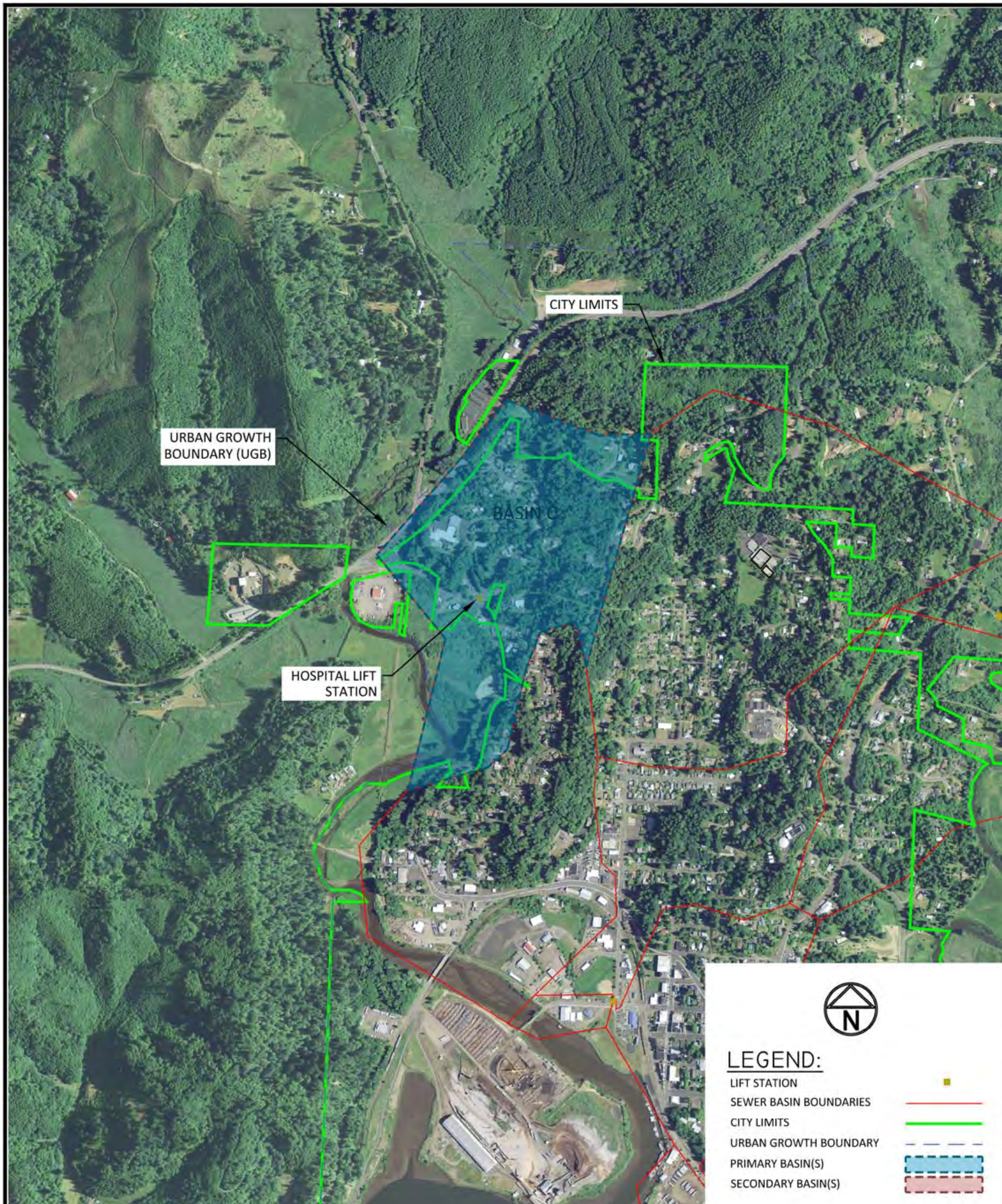
Backup power at the lift station is provided by an 80 KW Diesel Generator. The City has current plans and budget to equip the generator with an automatic transfer switch.

See Figures 4.2.4, 4.2.4.a, 4.2.4.b and 4.2.4.c for service basin, facility layout and schematics for the Lincoln Way Lift Station.

The forcemain between the Lincoln Way Lift Station and the discharge manhole is 6" in diameter and the material varies between Ductile Iron pipe and Asbestos Cement pipe. The forcemain is approximately 2,400 feet long and follows the alignment of Old Hwy 20 to the discharge manhole. The profile along the forcemain is continuously ascending at various slopes. This force main has an air injection system installed to address the long periods in the pumping cycle.

Noted deficiencies with the Lincoln Way Lift Station include:

- The lift station building is settling damaging the structure.
- No ability to bypass pump at the lift station.
- No ability to monitor pump station flows (no flow meter).
- No enclosure for a dedicated on site backup power supply.
- Air injection system is not operational.
- Dry well access is classified as a confined space under OSHA guidelines and requires notification and recording every entry into the drywell.




Civil West Engineering Services, Inc.

DRAWN BY: MLG
DATE: DEC, 2012

0 1"

WASTEWATER FACILITIES PLAN

**Hospital Lift Station Service Area**

CITY OF TOLEDO
LINCOLN COUNTY, OR

FIGURE 4.2.4

# LINCOLN WAY LIFT STATION

**DESIGN DATA**

| | |
|---|---|
| Location: | Lincoln Way and Frontage Road |
| Type: | Duplex Submersible |
| Wetwell: | Precast Concrete |
| Diameter: | 8 ft |
| Area: | 50 sf |
| Volume: | 376 gal/ft depth |
| | 1128 gal @ 3-ft range |
| Pump Type: | Constant Speed, Non-Clog |
| Capacity (each): | 290 gpm |
| | 0.42 MGD |
| Capacity (both): | 370 gpm |
| | 0.53 MGD |
| Pump HP (each) | 30 HP |
| Level Control Type: | Pressure Transducer |
| Overflow Point: | Manhole C-3 |
| Level Control Type: | Pressure Transducer |
| Overflow Discharge: | Ditch @ Frontage Rd to Depot Slough |
| Average Time to Overflow | |
| ADWF: | 0.05 MGD |
| Wetwell Volume: | 1128 gal @ 3-ft range |
| Influent Sewer Length: | 1800 ft |
| Inf. Sewer + MH Volume: | 5500 gal |
| Influent Sewer Invert: | 0.09 IE @ Wetwell |
| Wetwell Invert: | -8.00 IE Wetwell |
| Wetwell Overflow Elevation: | 11.00 EL TOS Wetwell |
| Overflow Manhole Elevation: | 8.00 Rim EL |
| Time to Overflow: | 6.60 Hours |
| Alarm Telemetry: | Autodialer |
| EPA Reliability Class: | Class I |

**CURRENT OPERATION SETTINGS**

| | |
|---|---|
| Wetwell Invert Elevation | -8.0 |
| Low Low Level (Alarm) | -6.0 |
| Low Level (Alarm) | -4.0 |
| Lead Pump Off | -3.5 |
| Lag Pump Off | -3.5 |
| Lead Pump On | -2.0 |
| Lag Pump On | -1.8 |
| High Level (Alarm) | -1.5 |
| High High Level (Alarm) | -1.0 |

**FORCE MAIN**

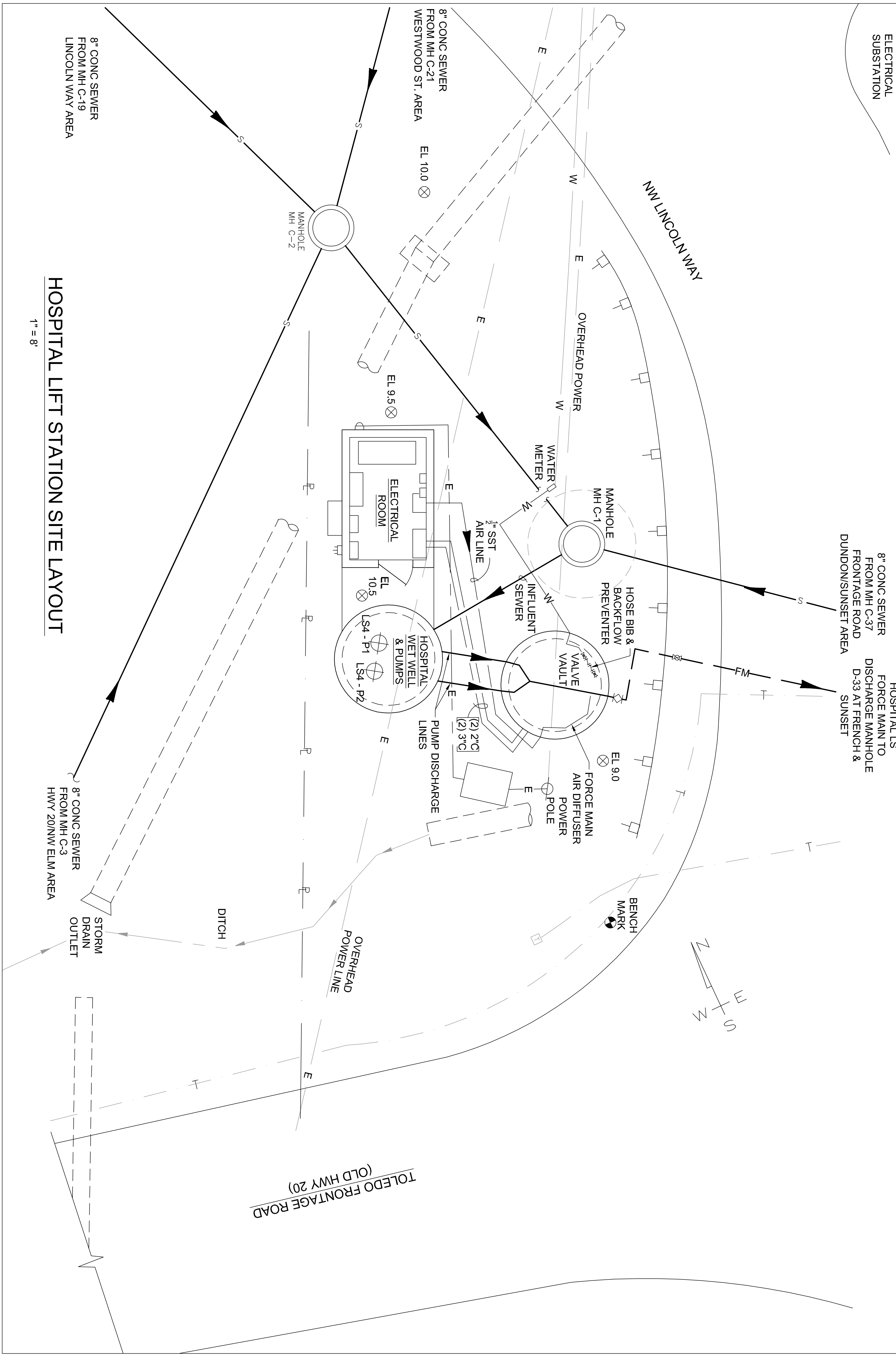
| | |
|---|---|
| Pipe Material: | Ductile Iron/Asbestos Cement |
| Length: | 2400 ft |
| Diameter: | 6 inch (0.20 sf) |
| Force Main Velocity | |
| (1) Pump: | 3.3 fps @ 290 gpm |
| (2) Pumps: | 4.2 fps @ 370 gpm |
| Detention Time: | 116 minutes |
| Volume - Force Main: | 3524 gallons |
| Volume - Wetwell: | 1128 gallons |
| Volume - FM+WW: | 4652 gallons |
| ADWF: | 35 gpm |
| FM Detention Time: | 12 min @ 3.3 fps |
| Profile: | Continuously ascending at |
| Discharge MH: | Manhole D-33 |
| Air/Vac. Release Valves: | Automatic Combination AVRV |
| Sulfide Control System: | Air Injection |

**AIR INJECTION SYSTEM**

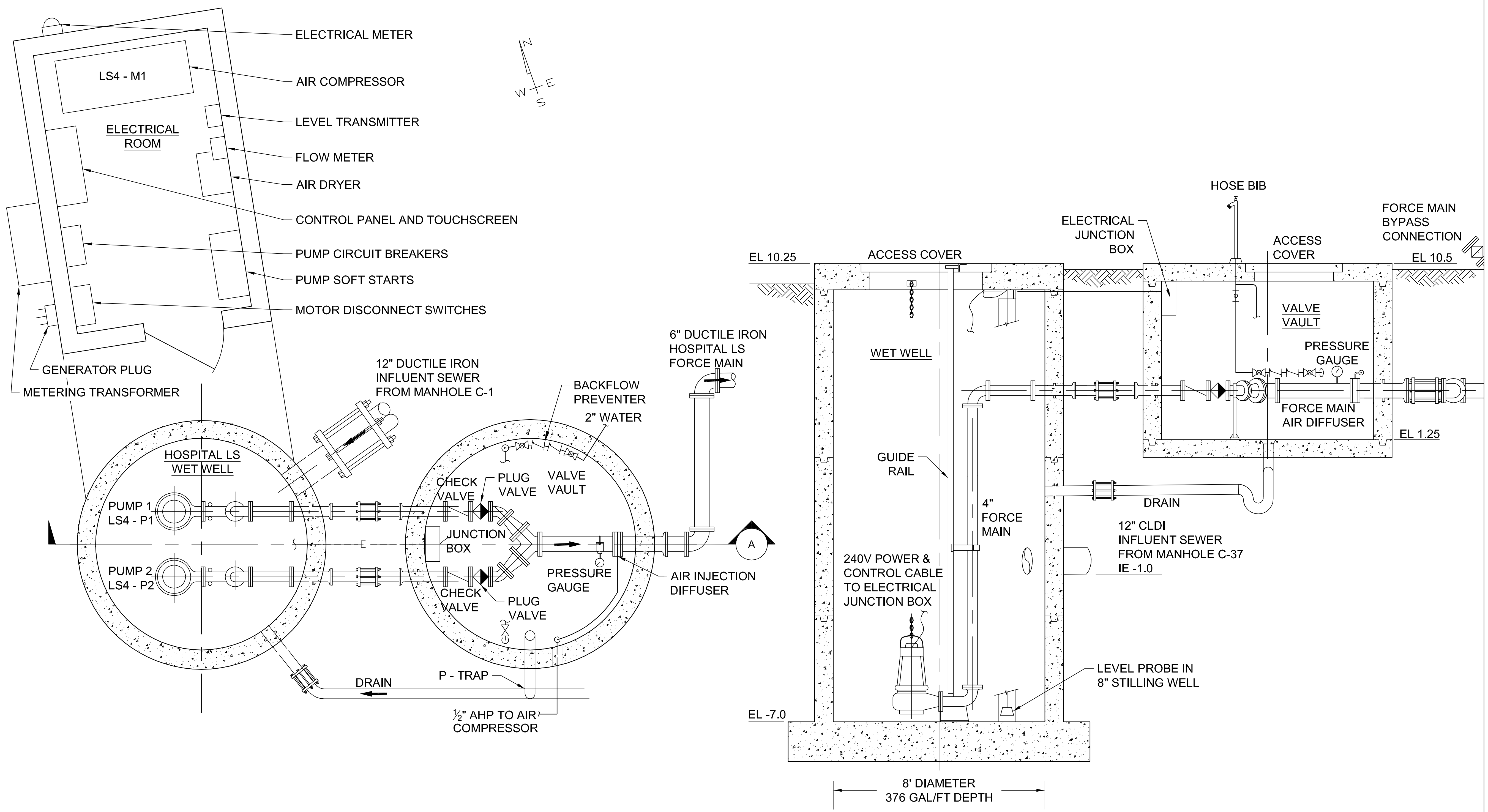
| | |
|---|---|
| Compressor HP: | 3 HP |
| Standard Injection Rate: | 0.15 SCFM |
| Actual Air Rate: | 0.02 cfm |
| Air Flowmeter Capacity: | 0.0 - 0.25 SCFM |
| Injector Type: | Ring |

**AUXILIARY POWER**

| | |
|---|---|
| Type: | Diesel Generator |
| Location: | On Site |
| Output: | 80 KW |
| Fuel Tank Capacity: | 50 gallons |
| Transfer Switch: | Manual |




| FIGURE 4.2.4b | 0 1" 1" = 8' Drawn By: MDW Date: Nov. 22, 2011 | **HOSPITAL LIFT STATION SITE PLAN** Wastewater Facilities Plan | City of Toledo Lincoln County, Oregon | Civil West Engineering Services, Inc. |
|---|---|---|---|---|




WET WELL AND VALVE VAULT PLAN

1" = 4'

SECTION A

1" = 4'

NOTES:

1. HOSPITAL LIFT STATION WAS COMPLETELY RECONSTRUCTED IN 2000.

2. SEE AS-BUILT DRAWINGS - TOLEDO SCHEDULE A PUMP STATION IMPROVEMENTS, *(CLEARWATER, 2000)*.

Civil West Engineering Services, Inc.

CITY OF TOLEDO
LINCOLN COUNTY, OREGON

**HOSPITAL LIFT STATION MECHANICAL PLAN**

WASTEWATER FACILITIES PLAN

0 1"
1" = 4'

DRAWN BY: MDW
DATE: NOV. 22, 2011

FIGURE 4.2.4c

### 4.2.5. Butler Bridge Lift Station

Butler Bridge Lift Station

The Butler Bridge Lift Station is located on the south side of Butler Bridge Road approximately one mile north of the bridge and serves Basins I, J, and K, including wastewater pumped by the A Street Lift Station. See Figure 4.2.5 for the Butler Bridge Lift Station Service area map. The lift station was originally constructed in 1955 and was upgraded in 1985, 1990, and 2000. The lift station has two, 100 horsepower, non-clog, variable speed pumps, which pump the wastewater to the headworks of the treatment plant. The design capacity of the lift station with one pump operating, as is normally the case, is 2160 gpm (3.11 mgd), and with both pumps on is 3125 gpm (4.5 mgd).

The pumps are set in a semicircular drywell, with the other half of the circle being the wetwell. The wetwell and drywell are approximately 20 feet deep, from the top of the concrete to the floor of the well. The wetwell has a volume of 853 gallons between the Lead Pump On elevation and the Lead Pump Off elevation (4').

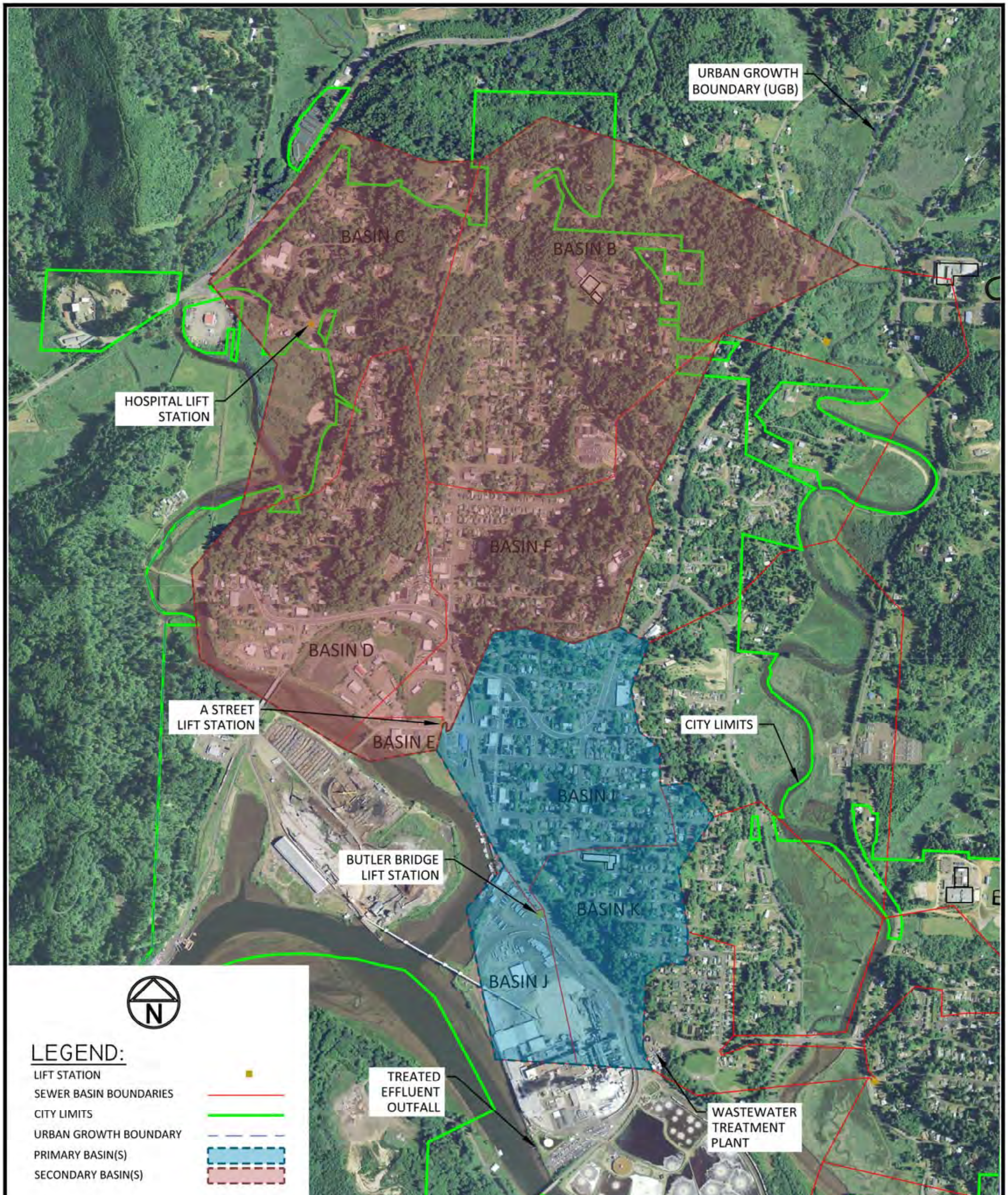
See Figures 4.2.5, 4.2.5.a, 4.2.5.b and 4.2.5.c for service basin, facility layout and schematics for the Butler Bridge Lift Station.

Backup power at the lift station is provided by an 100 KW Diesel Generator equipped with an automatic transfer switch.

The forcemain between the Butler Bridge Lift Station and the wastewater treatment plant is a combination of 14" Ductile Iron pipe installed in 1982 (~1400 feet) and 14" HDPE pipe installed in 2010 (~500 feet). The Ductile Iron forcemain runs southeast along Butler Bridge road to a point where, in 2010, newer HDPE pipe was attached and bored beneath the railroad tracks and up to the plant headworks. There is one Air Release Valve approximately 1040 feet south of the lift station.

Noted deficiencies with the Butler Bridge Lift Station include:

- The lift station building and generator enclosure is settling creating cracks in the ceiling and walls and prohibiting the doors from opening and closing correctly.
- The facility has had over-heating issues with the motors, VFDs, and other system controls.
- The partition wall separating the wet and dry wells is leaking.
- Dry well access is classified as a confined space under OSHA guidelines and requires notification and recording every entry into the drywell.




| | | | | |
|---|---|---|---|---|
| Civil West Engineering Services, Inc. | DRAWN BY: MLG<br>DATE: DEC, 2012 | 0 — 1" | **Butler Bridge Lift Station Service Area** | FIGURE |
| | WASTEWATER FACILITIES PLAN | | CITY OF TOLEDO<br>LINCOLN COUNTY, OR | 4.2.5 |

## DESIGN DATA

| | |
|---|---|
| Location: | Butler Bridge Road, 1 mile north of Bridge |
| Type: | Duplex, Dry Pit, Flooded Suction |
| Wetwell: | Concrete Split Caisson |
| Diameter: | 12 ft |
| Area: | 38 sf |
| Volume: | 284 gal/ft depth |
| | 853 gal @ 3-ft range |
| Pump Type: | Variable Speed, Non-Clog |
| Capacity (each): | 2,160 gpm |
| | 3.11 MGD |
| Capacity (both): | 3,125 gpm |
| | 4.50 MGD |
| Pump HP (each) | 100 HP |
| Level Control Type: | Pressure Transducer |
| Overflow Point: | Manhole J-1 |
| Level Control Type: | Catharine Street |
| Overflow Discharge: | Depot Slough |
| Average Time to Overflow | |
| ADWF: | 0.60 MGD |
| Wetwell Volume: | 853 gal @ 3-ft range |
| Influent Sewer Length: | 2,000 ft |
| Inf. Sewer + MH Volume: | 21,000 gal |
| Influent Sewer Invert: | -3.50 IE @ Wetwell |
| Wetwell Invert: | -11.08 IE Wetwell |
| Wetwell Overflow Elevation: | 8.50 EL TOS Wetwell |
| Overflow Manhole Elevation: | 8.50 Rim EL |
| Time to Overflow: | 0.80 Hours |
| Alarm Telemetry: | Autodialer |
| EPA Reliability Class: | Class I |

## CURRENT OPERATION SETTINGS

| | |
|---|---|
| Wetwell Invert Elevation | -11.0 ASL |
| Low Low Level (Alarm) | 400.0 |
| Low Level (Alarm) | 500.0 |
| Lead Pump Off | 675.0 |
| Lag Pump Off | 675.0 |
| Lead Pump On (Min Speed) | 860.0 |
| Lead Pump On (Max Speed) | 910.0 |
| Lag Pump On (Min Speed) | 1250.0 |
| Lag Pump On (Max Speed) | 1300.0 |
| High Level (Alarm) | 1600.0 |
| High High Level (Alarm) | 1650.0 |

Civil West Engineering Services, Inc.

DRAWN BY: MDW
DATE: NOV. 29, 2011

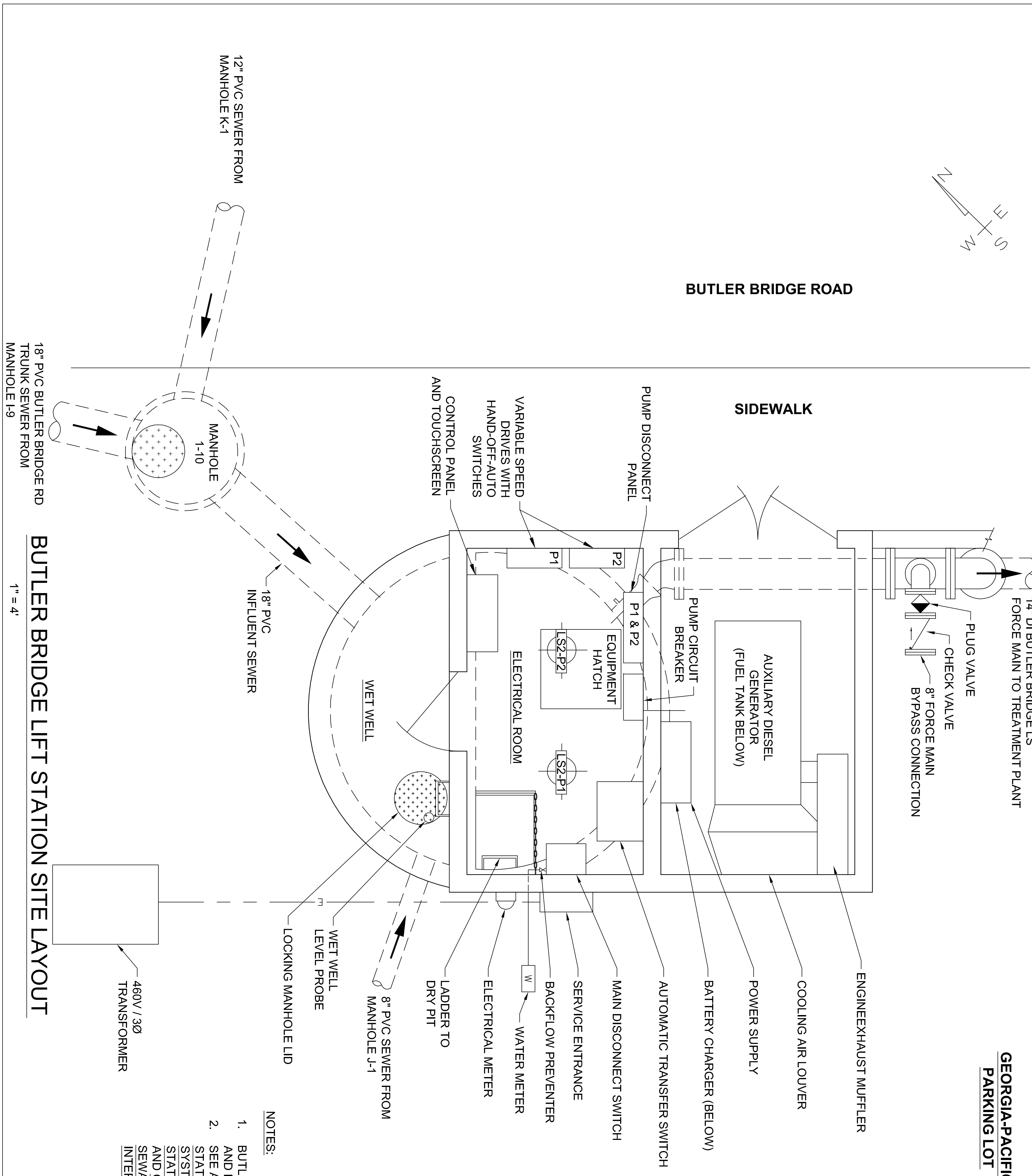
0 1"

WASTEWATER FACILITIES PLAN

**BUTLER BRIDGE LIFT STATION DESIGN DATA**

CITY OF TOLEDO
LINCOLN COUNTY, OR

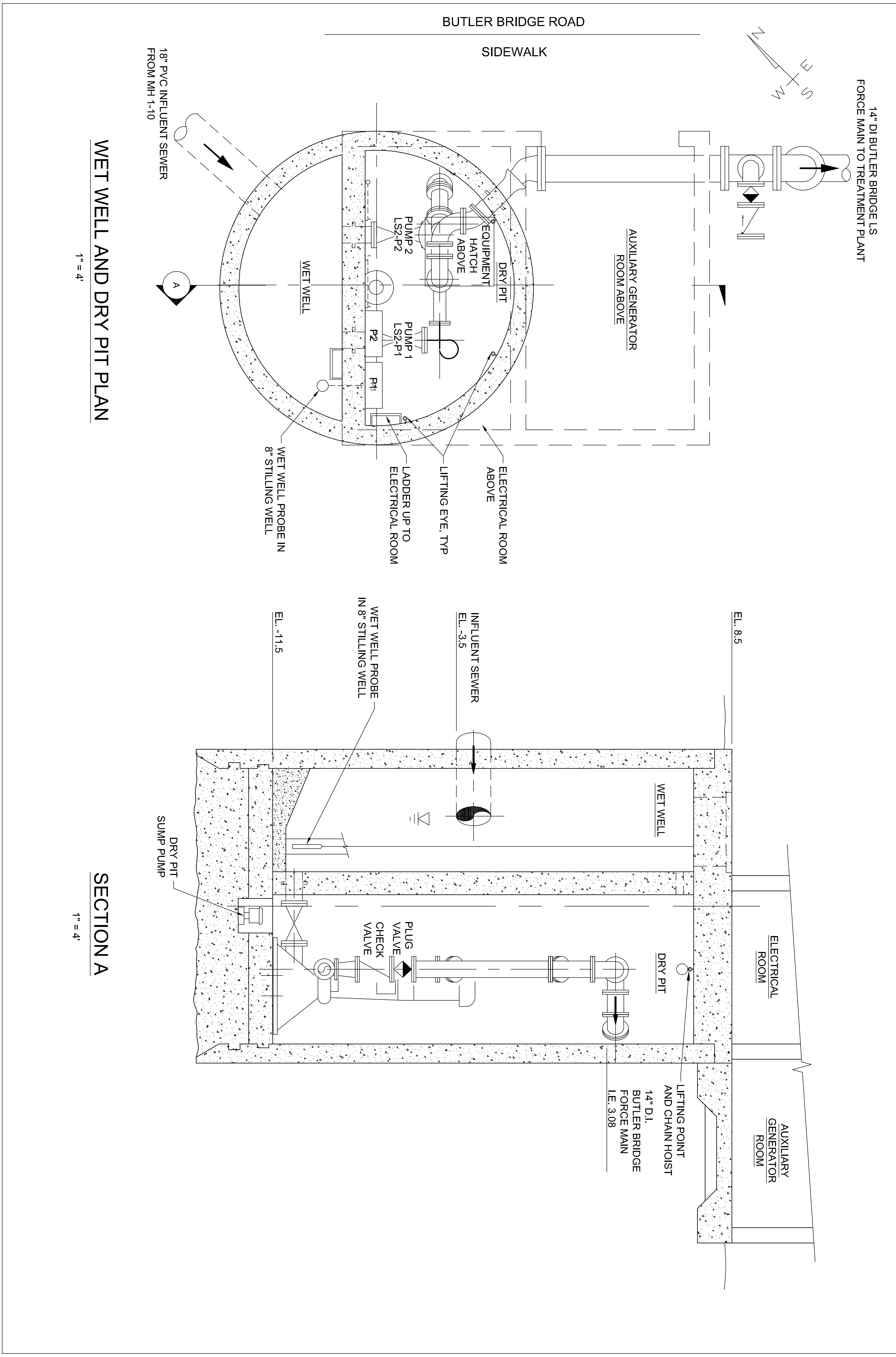
FIGURE 4.2.5a




NOTES:

1. BUTLER BRIDGE ROAD LIFT STATION WAS CONSTRUCTED IN 1955 AND REHABILITATED IN 1985 AND 2000.
2. SEE AS-BUILT DRAWINGS ENTITLED TOLEDO SCHEDULE A PUMP STATION IMPROVEMENTS (*CLEARWATER 2000*); SANITARY SEWER SYSTEM REHABILITATION (*WESTECH 1990*); SEWAGE LIFT STATION AND FORCE MAIN IMPROVEMENTS (*WESTECH 1981*); AND CONTRACT DOCUMENTS FOR THE CONSTRUCTION OF SEWAGE TREATMENT PLANT AND PUMPING STATIONS AND AN INTERCEPTOR SEWER (*CH2M, 1954*).

| | | | | |
|---|---|---|---|---|
| FIGURE 4.2.5b | 0 1" — 1" = 4' — DRAWN BY: MDW — DATE: NOV. 22, 2011 | **BUTLER BRIDGE LIFT STATION SITE PLAN** — WASTEWATER FACILITIES PLAN | CITY OF TOLEDO — LINCOLN COUNTY, OREGON | Civil West Engineering Services, Inc. |




| | | | | |
|---|---|---|---|---|
| FIGURE 4.2.5c | 0 1" 1" = 4' DRAWN BY: MDW DATE: NOV. 22, 2011 | **BUTLER BRIDGE LIFT STATION MECH. PLAN** WASTEWATER FACILITIES PLAN | CITY OF TOLEDO LINCOLN COUNTY, OREGON | Civil West Engineering Services, Inc. |

## 4.3. Existing Wastewater Treatment Plant

The existing Wastewater Treatment Plant, as it was originally constructed in 1954, included a primary clarifier (now Final Clarifier 1), an anaerobic digester, an effluent structure (abandoned), the 18" outfall (still in use) and the sludge drying beds south of the railroad tracks (abandoned).

In 1970, the City constructed a concrete contact stabilization package plant to provide the facility with secondary treatment capability, and upgraded the chlorine disinfection system.

In 1981, the City doubled the treatment plant capacity to 3.2 million gallons per day (mgd) by adding a headworks, a second contact stabilization unit and a second final clarifier. Improved chlorination and polymer addition facilities were provided.

In 2000, the Treatment Plant received a new headworks, a new secondary clarifier, a new two-cell digester, an expansion of the Treatment Unit 2 aeration basin, and various other site improvements. Currently the plant is designed to accept a short duration peak flow of 6.5 mgd. The headworks are sized to accommodate this peak flow, while the remainder of the plant is designed to operate at a maximum flow of 4.3 mgd. To account for the difference between the headworks capacity and the rest of the plant capacity is a 4,000 gallon equalization chamber which is built integral with the headworks and the old TU2 clarifier (~160,000 gallons) which serves as a surge basin to dampen the peak flows.

See Figure 4.3.a for a Site Plan of the current treatment facilities.
See Figure 4.3.b for process flow diagram.

### 4.3.1. **Headworks**

Included in the 1999/2000 plant improvements was a new headworks, see Figure 4.3.1. The headworks consist of two different Parshall flumes (a 12" flume to measure flows from the Butler Bridge Lift Station and a 9" flume to measure flows from the Ammon Road Lift Station and the gravity system serving Basin L). There is an inclined shaftless auger with 0.25" openings which serves as the primary screen and a manually cleaned bar rack with 0.5" openings as the standby/overflow screen.

Each screen (inclined shaftless auger and manually cleaned bar rack) is rated at 4.5 MGD, however operators have noted that during periods when the Butler Bridge Lift Station is pumping at a high rate, the influent will often "jump" the wall and go into the manually screened channel.

There is 10' diameter vortex grit basin, which has a rated capacity of 6.6 MGD. Included is a non-clog centrifugal (WEMCO CE) grit pump. The plant operators have not noted any concerns regarding the existing unit.

In 2012-13 the City installed a Pista grit classifier to replace the plant's old failing grit system. The new system includes a grit concentrator and a 9 inch diameter dewatering screw grit conveyor.

During the 2000 improvements and as part of the headworks structure a 4000 gallon equalization chamber was installed. The purpose of this chamber was to provide a relatively constant flow from the headworks which, because it is fed by two pump stations, naturally receives surges of flow. The design was intended to provide a floating outlet which would provide a constant flow of 0.4 MGD, however the outlet did not work properly and the operators have since removed it. The equalization chamber still mitigates surges, although the flows vary as the depth of liquid in the vault varies. The flow goes through the 6" pipe that was previously connected to the floating outlet and into the outlet box. During high flows, the flow overtops a weir directly into the outlet box. The aeration system originally installed in the equalization chamber has been disconnected since the storage time in the chamber is lower without the flow equalization device in place.

See Figure 4.3.1 for headworks plans.

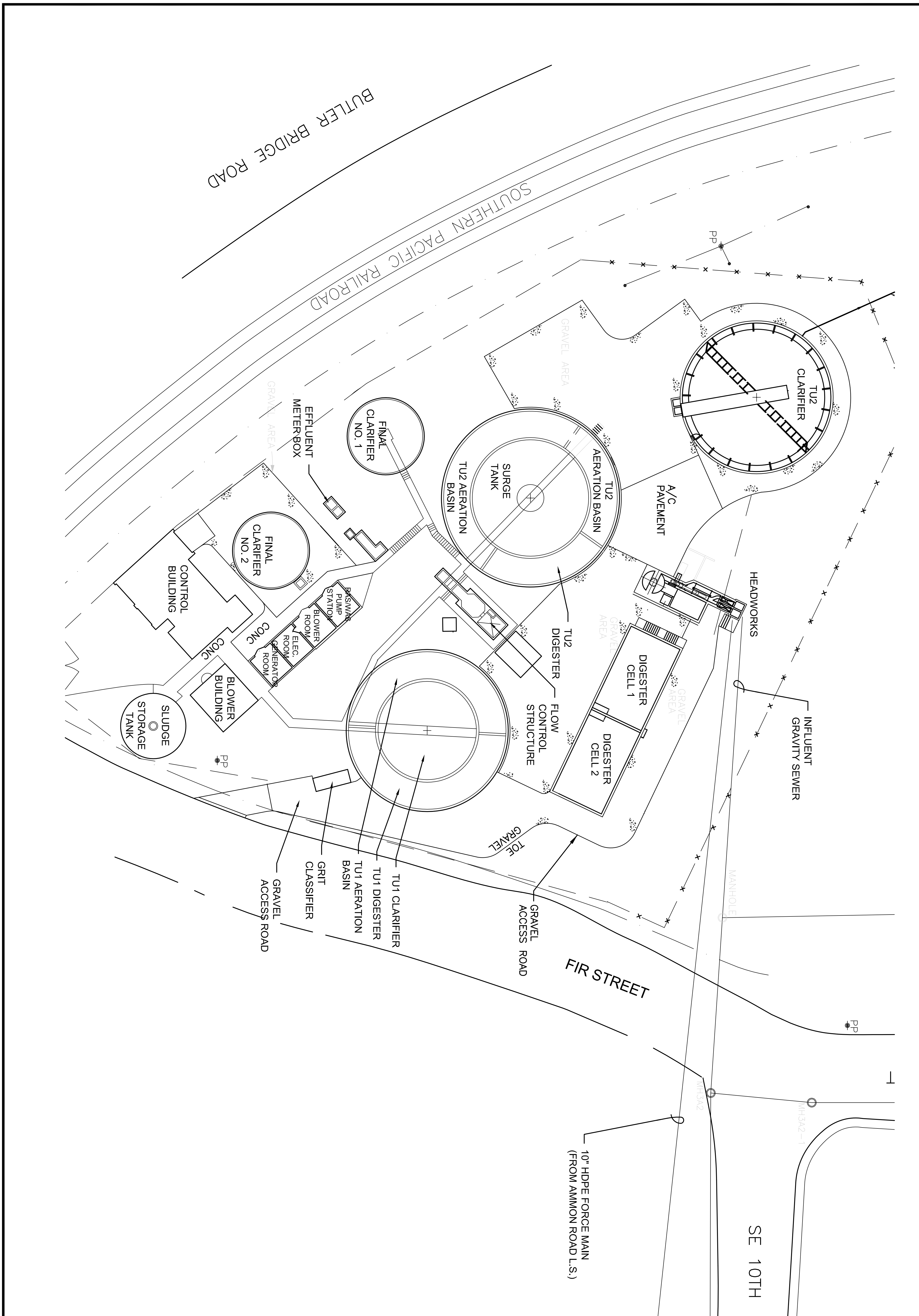



FIGURE 4.3a

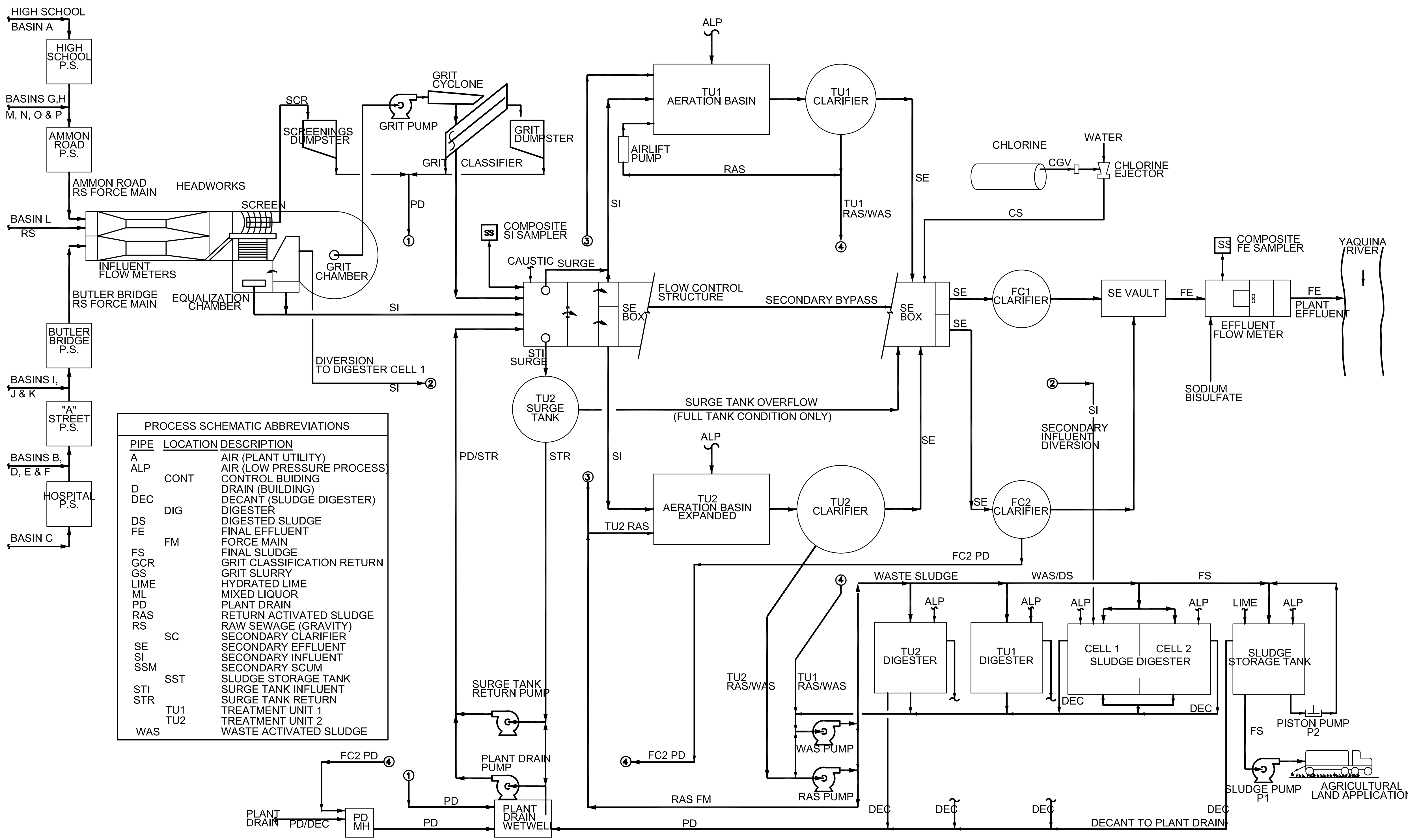
0 1"
SCALE VARIES

Drawn By: MDW
Date: Nov. 22, 2011

**WWTP SITE PLAN**

Wastewater Facilities Plan

City of Toledo
Lincoln County, Oregon




PROCESS SCHEMATIC ABBREVIATIONS

| PIPE | LOCATION | DESCRIPTION |
|---|---|---|
| A | | AIR (PLANT UTILITY) |
| ALP | | AIR (LOW PRESSURE PROCESS) |
| | CONT | CONTROL BUIDING |
| D | | DRAIN (BUILDING) |
| DEC | | DECANT (SLUDGE DIGESTER) |
| | DIG | DIGESTER |
| DS | | DIGESTED SLUDGE |
| FE | | FINAL EFFLUENT |
| | FM | FORCE MAIN |
| FS | | FINAL SLUDGE |
| GCR | | GRIT CLASSIFICATION RETURN |
| GS | | GRIT SLURRY |
| LIME | | HYDRATED LIME |
| ML | | MIXED LIQUOR |
| PD | | PLANT DRAIN |
| RAS | | RETURN ACTIVATED SLUDGE |
| RS | | RAW SEWAGE (GRAVITY) |
| | SC | SECONDARY CLARIFIER |
| SE | | SECONDARY EFFLUENT |
| SI | | SECONDARY INFLUENT |
| SSM | | SECONDARY SCUM |
| | SST | SLUDGE STORAGE TANK |
| STI | | SURGE TANK INFLUENT |
| STR | | SURGE TANK RETURN |
| | TU1 | TREATMENT UNIT 1 |
| | TU2 | TREATMENT UNIT 2 |
| WAS | | WASTE ACTIVATED SLUDGE |

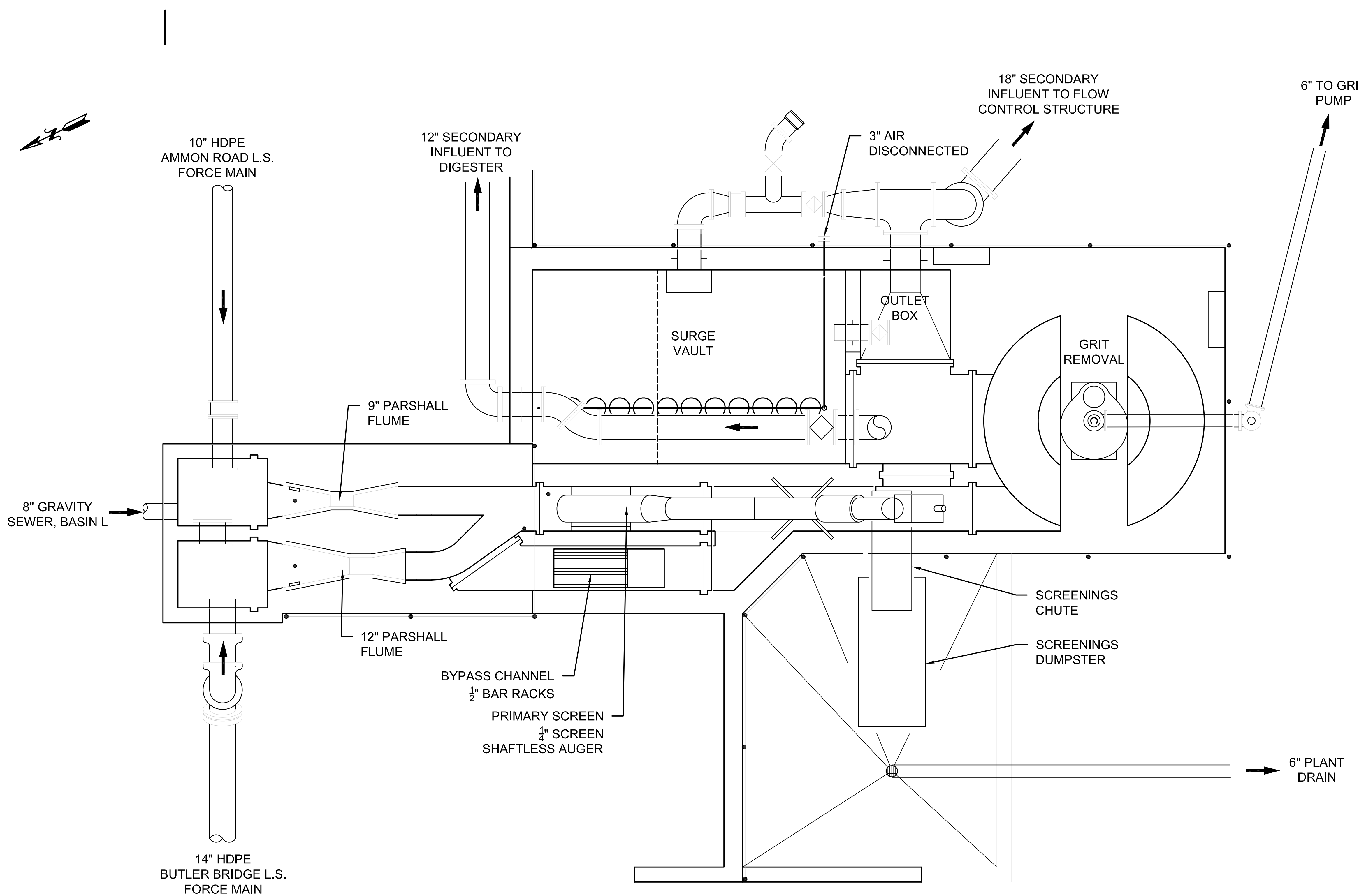
CITY OF TOLEDO
LINCOLN COUNTY, OREGON

# WWTP PROCESS FLOW DIAGRAM

WASTEWATER FACILITIES PLAN

0 1"
N.T.S.
DRAWN BY: MDW
DATE: JULY 2013

FIGURE 4.3b




City of Toledo
Lincoln County, Oregon

**WWTP HEADWORKS**

Wastewater Facilities Plan

0 1"
SCALE VARIES

Drawn By: MDW
Date: Nov. 22, 2011

FIGURE 4.3.1

### 4.3.2. Flow Control System

When the new headworks were constructed in 2001, the old headworks were transformed into a flow control structure. Influent is gravity fed from the new headworks into the north end of the flow control structure. Flow is then split between the two treatment units, with the default split being 36.5% to Treatment Unit 1 (TU1) and 63.5% going to Treatment Unit 2 (TU2). Lime is injected at this point to keep pH levels from dropping too low. Peak flows are also routed from the flow control structure to, and stored in the old TU2 clarifier. Stored flows are then pumped back into the flow control structure when flows subside.

See Figure 4.3.2 for flow control plans.

### 4.3.3. Aeration

Both treatment units consist of aeration basins around the perimeter of circular clarifiers. The TU1 aeration basin is around the perimeter of the TU1 clarifier. The TU2 aeration basin is around the old TU2 clarifier, which is now used as the surge tank. The design summary of the aeration basins is below:

<u>TU1 Aeration basin:</u>

| | |
|---|---|
| Type | Plug Flow Channel |
| Aeration | Fine Bubble Tube Diffusers |
| Peak Influent Flow | 1.5 MGD |
| Maximum RAS Flow | 120 gpm |
| Percent RAS at peak | 12% |
| Volume | 116,321 |
| Length | 90.6 feet |
| Width | 12.0 feet |
| Depth | 14.3 feet |

<u>TU2 Aeration basin:</u>

| | |
|---|---|
| Type | Plug Flow Channel |
| Aeration | Membrane Tube Diffusers |
| Peak Influent Flow | 2.8 MGD |
| Maximum RAS Flow | 910 gpm |
| Percent RAS at peak | 47% |
| Volume | 191,328 |
| Length | 153 feet |
| Width | 11.7 feet |
| Depth | 14.3 feet |

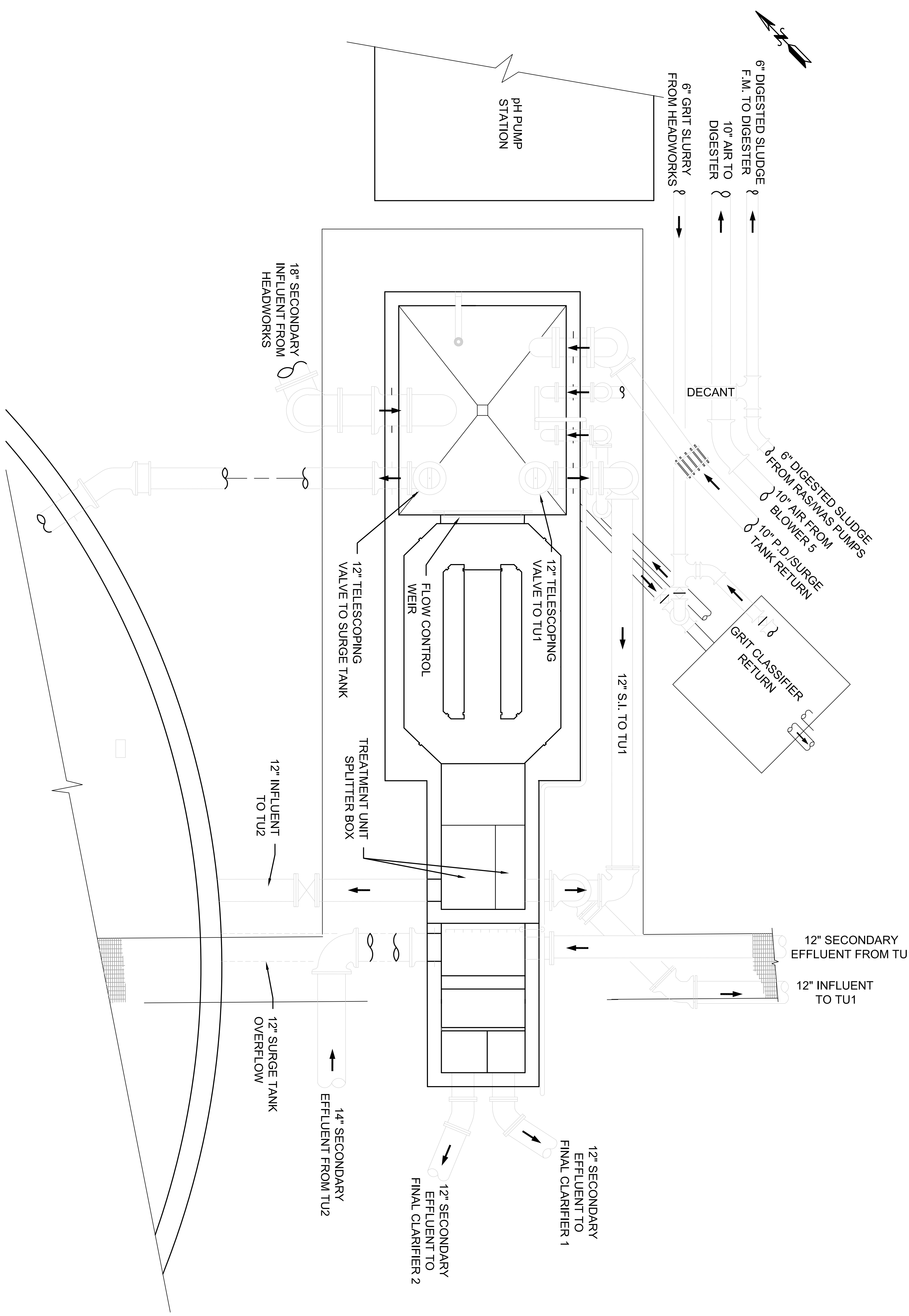



FIGURE 4.3.2

0 1"

SCALE VARIES

DRAWN BY: MDW
DATE: NOV. 22, 2011

**WWTP FLOW CONTROL**

WASTEWATER FACILITIES PLAN

CITY OF TOLEDO
LINCOLN COUNTY, OREGON

### 4.3.4. **Clarifiers**

Both treatment units flow from the aeration basins into circular clarifiers. The TU1 clarifier is original to the site and the TU2 clarifier was constructed new in 2001. The design summary of the clarifiers is below:

<u>TU1 Clarifier:</u>

| | |
|---|---|
| Type | Circular Concrete Tank<br>Peripheral Feed, Center Takeoff |
| Diameter | 44 feet |
| Sidewall Depth | 12.25 feet |
| Volume | 139,300 gallons |
| Area | 1,520 sf |
| Sludge Mechanism | Rake |
| RAS Pump | Airlift |
| WAS Pump | Airlift |
| Scum Pump | Airlift |
| Overflow Rate: | |
| ADWF | 4,000 gal/ft-day |
| AWWF | 7,200 gal/ft-day |
| PIF* | 24,300 gal/ft-day |

<u>TU2 Clarifier:</u>

| | |
|---|---|
| Type | Circular Concrete Tank<br>Peripheral Feed, Center Takeoff |
| Diameter | 66 feet |
| Sidewall Depth | 14.00 feet |
| Volume | 358,200 gallons |
| Area | 3,421 sf |
| Sludge Mechanism | Rake |
| RAS Pump | 910 gpm, 10 hp, Centrifugal Non-Clog |
| WAS Pump | 236 gmp, 5 hp, Centrifugal Non-Clog |
| Scum Pump | 1 hp, Submersible |
| Overflow Rate: | |
| ADWF | 4,000 gal/ft-day |
| AWWF | 7,200 gal/ft-day |
| PIF* | 24,300 gal/ft-day |

* - PIF overflow rates are based on maximum treated flow rate of 3.5 MGD

### 4.3.5. **Disinfection**

Effluent gravity flows from each of the clarifiers back to the lower portion of the flow control structure where chlorine is added. 12½% Hypochlorite, purchased by the city in 300 gallon “totes”, is metered into the effluent based on the flow measured at the effluent flow meter.

The flow is then split again and routed into one of two different final clarifiers (FC1 and FC2) to facilitate chlorine contact time. The two clarifiers have a combined 128,280 gallon capacity and flow is split evenly between them. Contact time in the final clarifiers is as follows:

- ADWF – 324 minutes
- AWWF – 177 minutes
- PDF – 47 minutes
- PIF – 28 minutes

Flow leaves the clarifiers and flows by gravity to the effluent metering box, where effluent is metered and a 25% solution of Sodium Bisulfate is added to remove any residual chlorine from the effluent.

Both chlorine and sodium bisulfate are metered based on the effluent flow meter. Injection rates are increased automatically as flow increases. During peak storm events, chlorine is adjusted manually to disinfect secondary treatment bypass flows. Per the existing O&M Manual, chlorine residual levels are tested often to ensure that the outfall does not exceed toxicity levels.

### 4.3.6. **Outfall**

After the flow is measured and dechlorinated, it flows by gravity through an 18" outfall to the Yaquina River. The outfall is located approximately 85 feet downstream of the Butler Bridge. The outfall is essentially a side-discharge pipe with a concrete headwall. The invert of the pipe is approximately 1.85 feet below MSL which means that during low low tides, the entire discharge pipe can be exposed. The discharge is a single port at River Mile 13.7. This area of the river is tidally influenced and the effluent mixing in the Yaquina River may be low during slack tide due to zero ambient velocities in the River.

The current permit provides for an allowable mixing zone (RMZ) that is that portion of the Yaquina River extending out one hundred feet from the east bank of the river and extending from a point one hundred feed upstream of the outfall to a point one hundred feed downstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within ten feet of the point of discharge.

The discharge pipe can be seen exiting the plant in Figure 4.3a, and further detail can be seen in the mixing zone study in Appendix B of this report.

### 4.3.7. **Sludge**

Activated sludge is generated during the treatment process and is either returned to the aeration basins as return activated sludge (RAS) or is thickened and stored as waste activated sludge (WAS). Sludge is collected from the TU1 and TU2 clarifiers and the RAS pump returns some of the sludge to the TU1 and TU2 aeration basin. The remainder of the sludge is pumped by the WAS pumps to a series of digesters. The plant has a digester on a portion the perimeter of the old TU1 clarifier, the remaining portion of the ring is the TU1 Aerator. Similarly, the TU2 clarifier is surrounded by a ring containing the TU2 aeration and more digester space. In addition, the 2001 improvements included the construction of a new, 200,000 gallon, digester. All of the digesters are complete mix, aerated type. After digestion, biosolids are stored in a 92,000 storage tank.

### 4.3.8. **Operations**

Unfortunately, the plant is not operable/operating as designed for several reasons, the most significant of which are noted below:

- The amount of sludge generated at the plant exceeds the capacity of the existing storage tank. The existing tank has a capacity of 92,000 gallons. During winter months, when sludge production exceeds the capacity of the tank, excess sludge is stored in the TU1 Aerator. This effectively removes TU1 from the treatment capacity of the plant, reducing the treatment capacity to 2.8 mgd.
- The effluent outfall pipe was original to the plant and does not have the capacity, most severely noted during high tides and high wastewater flows, to discharge the treated effluent as quickly as it is incoming. This can result in outfall bypass, or inefficient plant operation while maintaining a lower discharge. Plant operators recommend adding a pump station to pressurize the discharge.
- The flow equalization device which was intended to control the flows out of the surge vault did not operate acceptably and was removed. The intent of the flow equalization device was to provide a uniform flow to the flow control structure by “floating” an outlet on top of the liquid in the surge vault. During Peak flows, the influent would overflow the weir and go directly into the outlet box, bypassing the equalization device. Currently, all flow is routed through the 6” wall pipe (previously the pipe from the equalization float valve) from the surge vault and into the outlet box. When the 6” pipe is overwhelmed, flows overtop a weir to pass from the surge vault into the outlet box. The result of this revision is that flows to the flow control structure vary as influent flows vary. Activated sludge plants are sensitive to plug flows and do not operate as efficiently if the flow constantly varies like is currently the case.

# 5.0 Wastewater Flows

Section

5

## *5.1. Wastewater Volume*

The City of Toledo's Wastewater Treatment Plant is unique in that nearly all of the influent flow is directed from two lift stations. Therefore, the maximum flow into the plant is limited to the maximum pumping capacities of the two lift stations plus a relatively small amount of wastewater from the gravity line serving basin L.

### 5.1.1. **Flow Definitions**

Wastewater is typically described through flow and loading characteristics. Flow characteristics define the hydraulic volumes that the plant experiences and what it must be capable of treating. Loading characteristics describe what is in the wastewater (i.e. contaminants, waste products, chemicals, etc) that must be substantially removed before the water can be discharged into the environment as effluent.

*The following terms will be used in flow analysis and flow projections in this Study*:

Dry Weather Period: Defined as the period when the precipitation and streamflows are low. This period is defined in the Oregon Administrative Rules (OAR 340-041-0207) as May 1 through October 31.

Wet Weather Period: Defined as the period when streamflows, rainfall and groundwater levels are high. This period is defined in OAR 340-041-0207 as November 1 through April 30.

Average Annual Flow (AAF) or Average Daily Flow (ADF): Total wastewater flow for an average 12-month period, from January 1 through December 31, divided by the total number of days in the year.

Base Sewerage: Total daily flow for the period between June 1 and September 31. This is used as a basis to calculate I/I.

Average Dry-Weather Flow (ADWF): Total wastewater flow for the dry-weather period divided by the number of days in the period.

Maximum Month Dry-Weather Flow (MMDWF): Total wastewater flow for the month with the highest flow during the dry-weather period, divided by the number of days in the month.

Average Wet-Weather Flow (AWWF): Total wastewater flow for the wet-weather period divided by the number of days in the period.

Maximum Month Wet-Weather Flow (MMWWF): Total wastewater flow for the month with the highest flow during the wet-weather period, divided by the number of days in the month.

Peak Day Average Flow (PDAF): Total flow for the day with the highest wastewater flow during the year.

Peak Week Flow (PWF): Average Daily Flow during the peak 7-day flow period.

Peak Instantaneous Flow (PIF): Flow for the highest peak of the year, expressed as a daily flow.

*The following terms will be used in the statistical analysis of flow rates:*

Ten-year Maximum Month Dry-Weather Flow ($MMDWF_{10}$): The monthly average dry-weather flow with a 10% probability of occurrence.

Five-year Maximum Month Wet-Weather Flow ($MMWWF_5$): The monthly average wet-weather flow with a 20% probability of occurrence.

Five-year Peak Day Average Flow ($PDAF_5$): The peak day average flow associated with a five-year storm event.

Five-year Peak Instantaneous Flow ($PIF_5$): The peak instantaneous flow during a five-year storm event.

*The following terms will be used in the Inflow and Infiltration Analysis:*

Base Infiltration Flow The base daily average flow in the wastewater collection system due to inflow and infiltration. It is calculated by subtracting the base sewer flow rate from the average dry-weather flow.

Average Wet-Weather Inflow and Infiltration Flow (AWW I/I) The daily average flow in the wastewater collection system due to inflow and infiltration. It is calculated by subtracting the base sewer flow rate from the average wet-weather flow.

Maximum Monthly Wet-Weather Inflow and Infiltration Flow (MMWW I/I) The average daily flow during the maximum monthly occurrence in the wastewater collection system due to inflow and infiltration. It is calculated by subtracting the base sewer flow rate from the system maximum monthly wet-weather flow.

Peak Day Inflow and Infiltration Flow (PD I/I) The maximum daily flow in the wastewater collection system due to inflow and infiltration. It is calculated by subtracting the base sewer flow rate from the system peak daily average flow.

Peak Instantaneous Inflow and Infiltration Flow (PIF I/I) The peak instantaneous or peak hourly flow in the wastewater collection system due to inflow and infiltration. It is calculated by subtracting the base sewer flow rate from the system peak instantaneous flow.

### 5.1.2. Summary of Available Data

The influent flow data included in the Discharge Monitoring Reports (DMRs) from January 2006 through July of 2011 have been used for flow analysis and wastewater characteristics. Influent flows can be measured by individual Parshall flume flow meters in the headworks (one to measure flow from Butler Bridge Lift Station and another to measure the combined flows from Ammon Road Lift Station and the gravity flow from basin L), however these flows have historically not been recorded. Treatment Plant flows, as recorded on the DMRs, are measured at the effluent flow control box with an 18” Water Specialties Propeller Meter.

Daily rainfall totals were referenced from the Wastewater Plant daily records.

Based on the DMR data described above, some of the design flows can be calculated. Below is the calculation AAF, Base Sewerage, ADWF, AWWF:

$$AAF = \frac{Average\ Total\ Wastewater\ Flow}{Days\ in\ Year} = \frac{291.92MG}{365.25\ Days} = 0.80\ MG/Day$$

$$Base\ Sewerage = \frac{Average\ Total\ Flow\ During\ Jun.-Sept.}{Days\ in\ Jun.-Sept.} = \frac{56.12\ MG}{122\ Days} = 0.46\ MGD$$

$$ADWF = \frac{Average\ Total\ Flow\ During\ Dry\ Period}{Days\ in\ Dry\ Period} = \frac{95.68\ MG}{184\ Days} = 0.52\ MGD$$

$$AWWF = \frac{Total\ Flow\ During\ Wet\ Period}{Days\ in\ Wet\ Period} = \frac{192.12\ MG}{181.25\ Days} = 1.06\ MGD$$

### 5.1.3. **Dry Weather Flow**

As indicated in the referenced DEQ guidelines, the ten-year Maximum Monthly Average Dry-Weather Flow ($MMDWF_{10}$) would be the monthly average flow in the rainiest summer month of high groundwater. West of the Oregon Cascades, the $MMDWF_{10}$ almost invariably occurs in May. The 10-Year MMDWF represents the anticipated monthly flow corresponding to the monthly rainfall accumulation during May with a 10% probability of occurrence in any given year.

Precipitation probabilities for various locations in Oregon are included in the report entitled *"Climatography of the United States No. 20, Monthly Station Climate Summaries, 1971 – 2000"* as published by the National Climatic Data Center. The closest probabilistic data sets are for the City of Newport and have been used for this analysis.

The graph in Figure 5.1.3 is based on five data points representing the average daily wastewater flows versus average monthly rainfall totals as shown in Table 5.1.3. The points generate a trend line which can be used to predict average wastewater flows from a given monthly rainfall total. The 10-year MMDWF is the flow corresponding to the 10% probability precipitation of 6.47 inches for the month of May, as determined by the above referenced climatography report. As shown in Figure 5.1.3, the corresponding $MMDWF_{10}$ is 0.86 MGD.

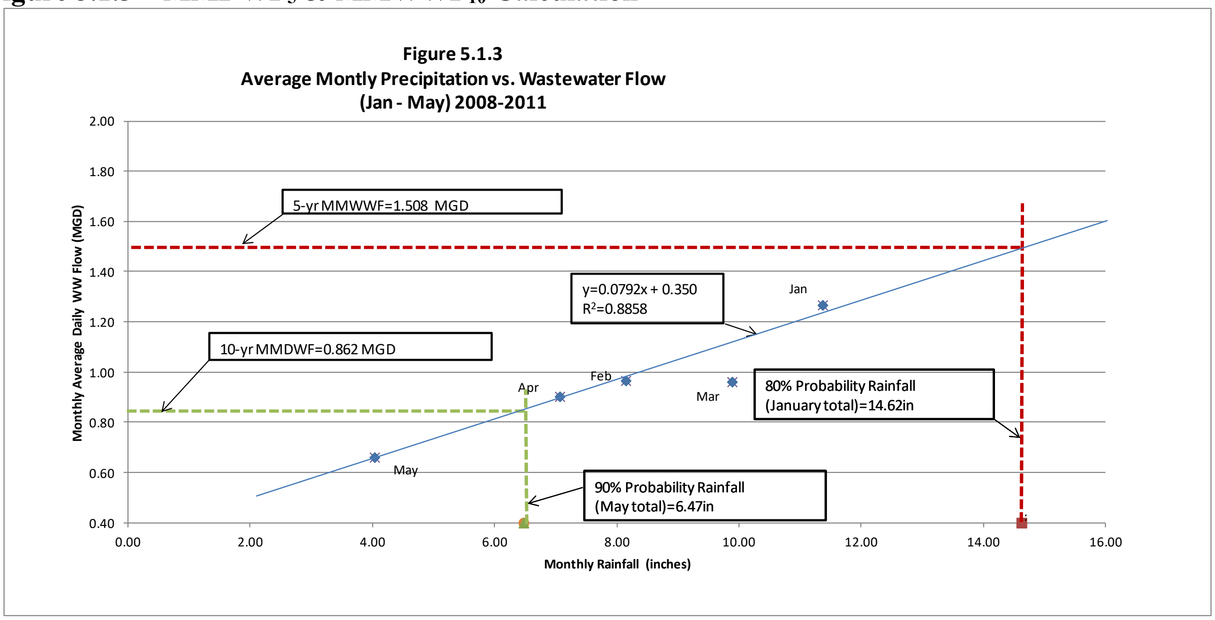
Table 5.1.3 also indicates the 10 year May accumulation (0.9 May) based on Data from *Climatology of the US No. 20 for years 1971-2000* published by the National Climate Data Center. This represents the amount which exceeds 9 out of 10 totals which have been recorded in May. It also indicates the 5 year January accumulation (0.8 Jan) which represents the amount which exceeds 4 out of 5 totals which have been recorded in January.

**Table 5.1.3 - Average Rainfall and Wastewater Flows**

| Precipitation and Rainfall Averages | | |
|---|---|---|
| Month | Monthly Rainfall (in/Mo) | Montly Avg. Day Flow (MGD) |
| Jan | 11.36 | 1.27 |
| Feb | 8.14 | 0.97 |
| Mar | 9.88 | 0.96 |
| Apr | 7.06 | 0.90 |
| May | 4.03 | 0.66 |
| 0.8 (Jan) | 14.62* | |
| 0.9 (May) | 6.47* | |

*Data from Climatology of the US No. 20 for years 1971-2000 published by the National climate Data Center

**Figure 5.1.3 – $MMDWF_5$ & $MMWWF_{10}$ Calculation**




## 5.1.4. **Wet Weather Flow**

Like many communities in western Oregon, the City of Toledo struggles with high volume wastewater flows caused by inflow and infiltration into the sanitary sewer system during the wet season. The flow analysis presented in the following section is based on the *Oregon DEQ Guidelines for Making Wet-Weather and Peak Flow Projections for Sewage Treatment in Western Oregon* (first published in 1996). These guidelines describe a detailed method for estimating wet-weather flow and peak flows in wastewater collection systems. This method is used to develop the minimum estimate for current flows from which to project future flow rates.

The referenced DEQ design guidelines indicate that high groundwater, west of the Cascades, is usually not attained until January, and heavy storms generally do not begin to cause a reliable or consistent infiltration response until January. Therefore, the MMWWF is expected to occur in January. The five-year January accumulation of 14.62 inches is indicated in the Climatography report based on rainfall probability data for Newport. When plotted with actual recorded events, the current five-year MMWWF is calculated to be 1.51 MGD (1048 gpm) as shown in Figure 5.1.3, above.

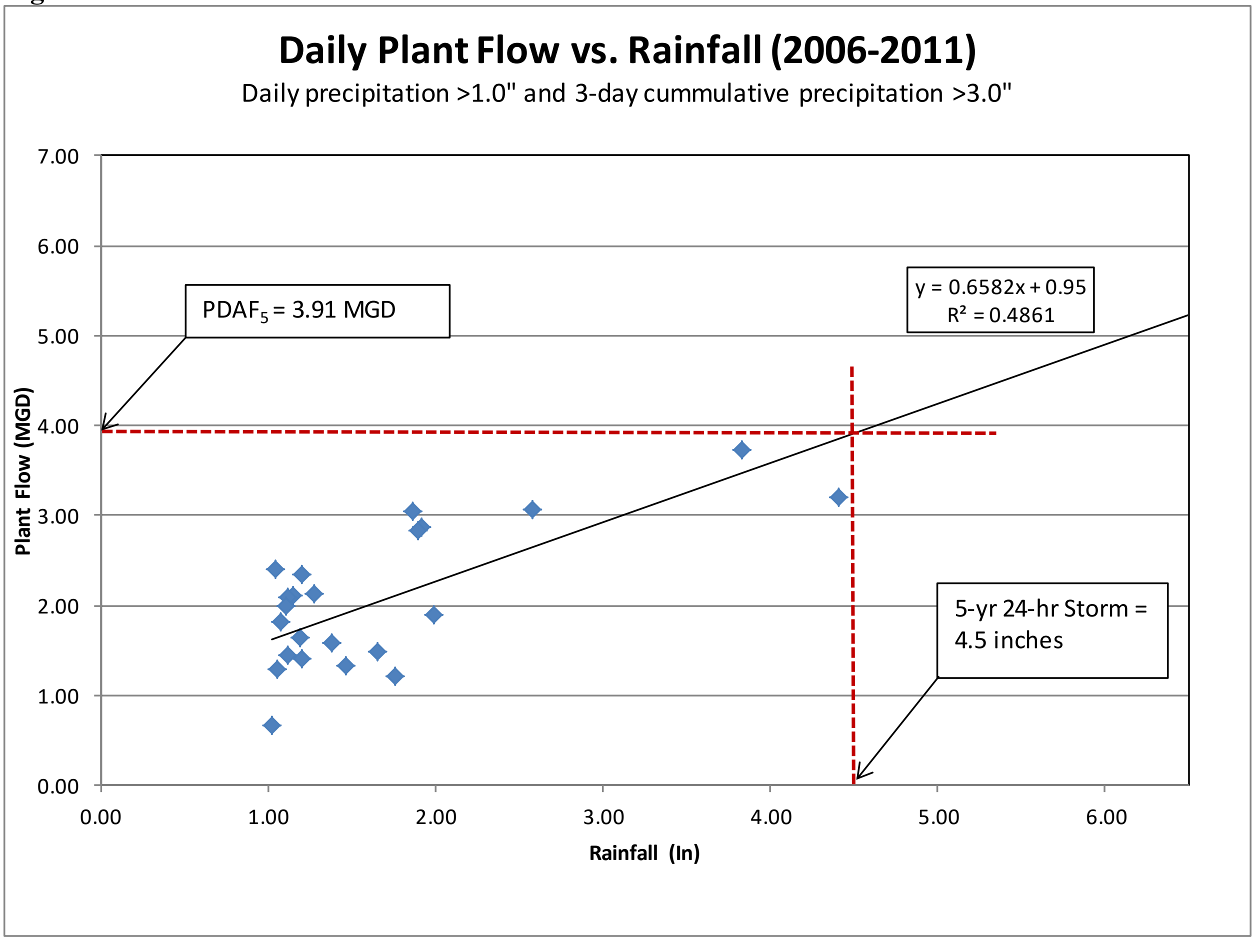
The Peak Day Average Flow ($PDAF_5$) corresponds to the five-year 24-hour storm event as defined by the NOAA isopluvial maps. Based on the NOAA maps, the five-year 24-hour event for the Toledo area is 4.5 inches of rain.

To determine the $PDAF_5$ using the DEQ methodology, actual events are plotted and a best-fit trendline is used to approximate the character of the system under different rainfall events. As in the graph above, rainfall data from the years 2006 through 2011 is used in the $PDAF_5$ calculation. Data points were selected based on the criteria that the daily rainfall was in excess of 1.0 inches and the 3-day cumulative (including event) rainfall was in excess of 3.0 inches. A summary of the data points used are included in Table 5.1.4 below. Results are graphed in Figure 5.1.4a.

**Table 5.1.4 – Significant Wet-Weather Rainfall and Flow Data**

| Daily Rainfall and Cooresponding Wastewater Flow (2006 - 2011) | | | | | |
|---|---|---|---|---|---|
| **Date** | **WW FLOW** (MGD) | **RAINFALL** (Inches) | **Date** | **WW FLOW** (MGD) | **RAINFALL** (Inches) |
| 7-Jan-06 | 2.14 | 1.27 | 3-Dec-07 | 3.20 | 4.41 |
| 10-Jan-06 | 3.06 | 2.58 | 4-Jan-08 | 1.49 | 1.65 |
| 8-Mar-06 | 1.41 | 1.20 | 6-Jan-08 | 1.64 | 1.19 |
| 3-Nov-06 | 0.66 | 1.02 | 30-Jan-08 | 2.00 | 1.11 |
| 5-Nov-06 | 1.89 | 1.99 | 31-Jan-08 | 2.10 | 1.15 |
| 7-Nov-06 | 3.73 | 3.83 | 1-Feb-08 | 1.45 | 1.12 |
| 12-Nov-06 | 1.81 | 1.08 | 2-Feb-08 | 3.04 | 1.86 |
| 23-Nov-06 | 2.09 | 1.12 | 12-Nov-08 | 2.82 | 1.89 |
| 27-Feb-07 | 2.40 | 1.04 | 8-Jan-09 | 2.87 | 1.92 |
| 17-Nov-07 | 1.20 | 1.76 | 12-Mar-10 | 1.57 | 1.38 |
| 18-Nov-07 | 1.33 | 1.46 | 1-Mar-11 | 2.35 | 1.20 |
| 19-Nov-07 | 1.30 | 1.05 | | | |

**Figure 5.1.4a – PDAF Calculation**



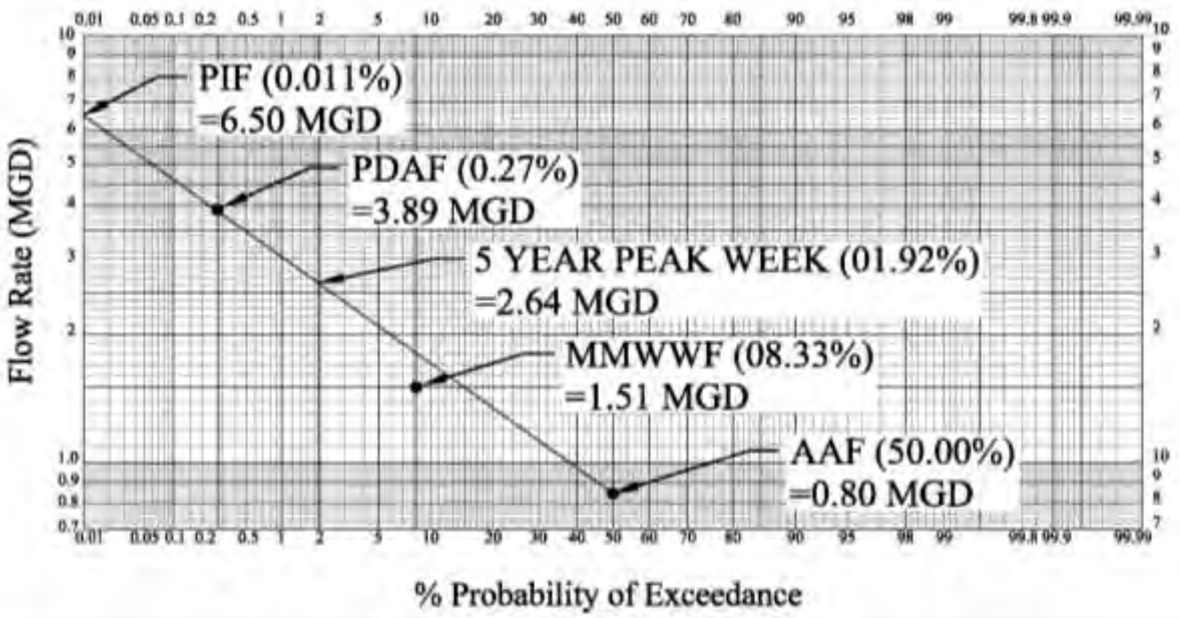

Based on Figure 5.1.4a, the current $PDAF_5$ is approximately 3.91 MGD (2714 gpm). This corresponds reasonably well with the plant DMR data.

DEQ guidelines for wastewater treatment plant design require critical plant and lift station components to be sized for the projected peak instantaneous flow ($PIF_5$). The current $PIF_5$ and 5-year peak week flow for the City of Toledo has been estimated using a probability graph on logarithmic probability paper based on the data summarized below:

- The average annual flow (AAF) has a probability of exceedance on any given day of 50%. AAF = 0.80 MGD
- The $MMWWF_5$, as determined in Figure 5.1.3, has a probability of exceedance of 1/12, or 8.33%. $MMWWF_5$ = 1.51 MGD.
- The peak week flow occurs one week out of the year, for a probability of exceedance of 1/52, or 1.92%.
- The $PDAF_5$ is the daily flow associated with the 5-year storm. The probability of exceeding the PDAF is 1/365, or 0.27%. As determined in Figure 5.1.4a, the $PDAF_5$ is 3.22 MGD.
- The PIF, or "peak hourly flow" occurs once per year for a probability of exceedance of: $\frac{1\ hour}{year} * \frac{1\ year}{365\ days} * \frac{1\ day}{24\ hours} = \frac{1}{8760} = .011\%$

Assuming, as allowed by the DEQ guidelines, that the maximum PIF occurs during the peak day, peak weak and peak month, we can create the graph shown below in Figure 5.1.4b.

**Figure 5.1.4b - PIF Calculation**




As shown above, when the known flow amounts and probabilities are plotted on a probability x 2 logarithmic graph, and a best fit trendline is added, unknown flows can be interpolated. In this way, the 5-year Peak Week Flow (2.64 MGD) and the PIF (6.50 MGD) are determined. However, based on the discussion at the beginning of Section 5, the maximum flow which can be received by the treatment plant is a function of the pumping capacities of the Butler Bridge Lift Station (4.5 MGD) and the Ammon Road Lift Station (2.0 MGD), plus a small amount of flow from Basin L. Because the flows used to determine these peak flows are at the plant discharge, there may be some inherent errors if any of the pump stations were unable to pump the true flow which would cause a measurement less than the actual flow.

5.1.5.**Infiltration and Inflow**

Nearly all coastal communities in Oregon struggle with the issue of inflow and infiltration (I/I) within their wastewater collection systems. Inflow and infiltration are defined as follows:

> <u>Infiltration</u>: Flows that enter the collection system through underground paths. Infiltration can be caused by high groundwater levels, rain-induced groundwater, and other sources. Infiltration flows make their way into the collection system through cracks in pipe, open or offset pipe joints, broken piping sections, leaks in manholes, and other below-grade openings in the collection system.
>
> <u>Inflow</u>: Flows that enter the collection system through above ground paths. Inflow is often related to building downspouts being connected to sanitary sewer service laterals, cross connections with storm drain systems that have not been separated, water flowing over manholes and entering in through the openings in the lids, catch basins, or area drains being connected to the sewer system, and other surface water sources.

When combined, Infiltration and Inflow (I/I) can result in tremendous increase in flows during the winter, particularly during prolonged storm events. Comparison of the records of daily rainfall and the WWTP flows shows a marked increase in wastewater inflow rates during heavy rain events. Current I/I levels can be summarized in the following table.

**Table 5.1.5 - Inflow / Infiltration Summary**

| Current I/I Flow Summary | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **Item** | | **Calculation** | | | | | **I/I Flow** | | | **Per Capita** |
| AWW I/I | = | AWWF -Base Sewerage = | 1.04 | - | 0.51 | = | 0.53 | MGD | = | 143.29 gppd |
| MMWW I/I | = | MMWWF - Base Sewerage = | 1.51 | - | 0.51 | = | 1.00 | MGD | = | 268.60 gppd |
| PD I/I | = | PDAF - Base Sewerage = | 3.91 | - | 0.51 | = | 3.40 | MGD | = | 914.33 gppd |
| PIF I/I | = | PIF - Base Sewerage = | 6.50 | - | 0.51 | = | 5.99 | MGD | = | 1610.56 gppd |

The City of Toledo commissioned an all inclusive Inflow and Infiltration Study. The results of that study are presented in the *City of Toledo, Inflow and Infiltration Study,* (2011, Civil West Engineering Services, Inc.). Three distinct survey projects were authorized by the City and completed by Civil West in order to pinpoint the major sources of I/I into the conveyance system. A smoke testing survey that was conducted during the dry summer months revealed faulty openings of the conveyance system to surface water. A flow mapping survey completed during wet winter months revealed areas where subsurface water leaks into the system. Finally, a television survey was conducted by inserting a small robotic camera into selected sewage manholes and pipelines.

Smoke testing identified nearly 200 individual collection system potential deficiencies. Flow mapping discovered 10 pipeline segments and several manholes experiencing high infiltration. Television inspection verified 18 pipe segments needing repair or replacement and identified many additional manholes showing signs of active or recent leaks. The final study recommended numerous improvement projects and provided cost estimates for each area to the City.

Based on the EPA I/I Analysis and Project Certification publication (#97-03), the determination of "non-excessive" INFILTRATION is based on an average flow rate during a period of seasonal high groundwater. For the purposes of this analysis, a period (March 13 through March 20) in 2010 was identified as having high ground water and little rain. The average flow during those 8 days was 0.94 MGD. Converting 0.94 MGD to a per capita flow rate is done by dividing by the population served (3,465 persons). Performing this calculation leads us to a daily per capita flow rate of 271 gpcd. This is above the EPA maximum rate. Therefore, per the EPA publication, the City of Toledo may have excessive infiltration.

Per the same EPA publication, excessive INFLOW is determined by the "highest daily flow recorded during a storm event". By this definition, the comparison should be made to the peak day average flow (PDAF). If the wet weather flow is below 275 gpcd, the inflow is considered non-excessive. The peak day average flow per capita for Toledo, as determined in Figure 5.1.4a is 3.91 MGD. Dividing by the current population (3,465 persons) we get a flow rate of 1128.43 gpcd. This is well in excess of the limit (275 gpcd) presented by the EPA. Therefore, per the EPA publication, the City of Toledo may have excessive inflow.

In addition to the I/I Study mentioned above, the City has performed some flow mapping in the lower areas to determine if a significant amount of brackish water from Yaquina Bay was entering the pipes. Surprisingly, the mapping indicated very little inflow in this area.

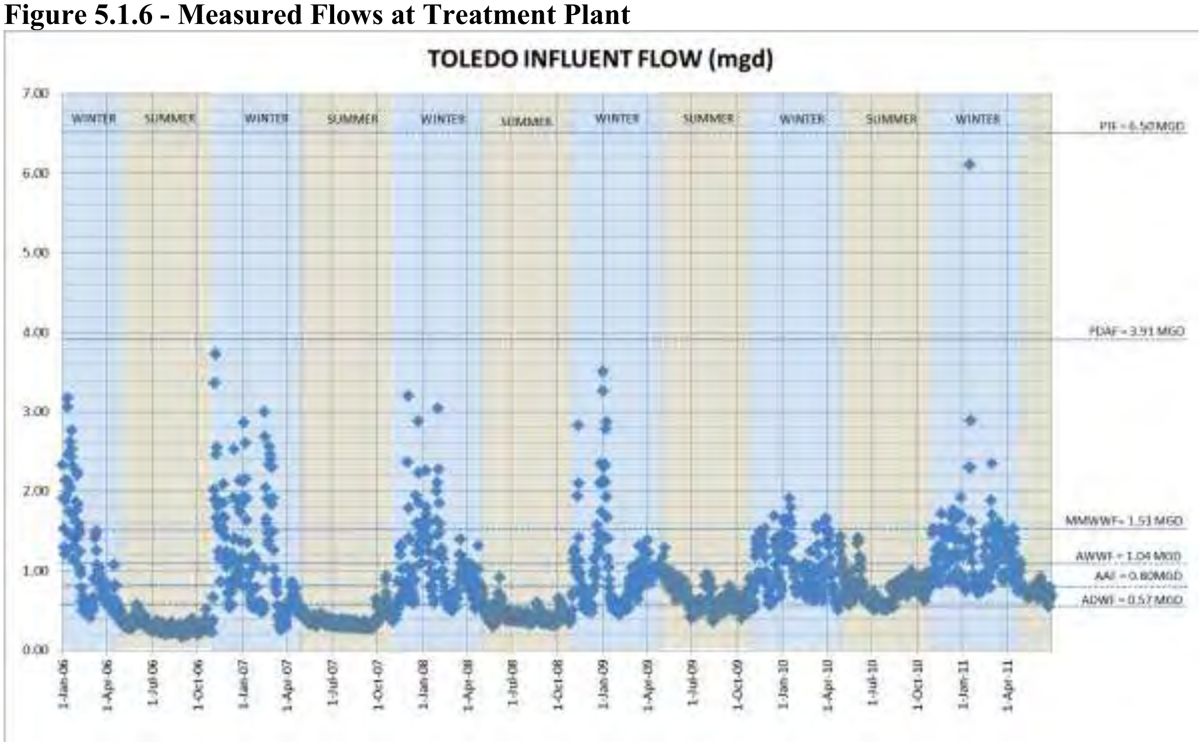
### 5.1.6. Summary of Existing Flows

Table 5.1.6 below summarizes the current dry and wet weather flows for the City of Toledo. Figure 5.1.6 shows a graph of the historical daily flows for the investigated 5 year period with the peak flow values identified. Definitions for the different flow criteria are provided in Section 5.1.1.

**Table 5.1.6 - Existing Wastewater Flow Summary**

| Summary of Current Wastewater Flows | | | |
|---|---|---|---|
| **Parameter** | **2010 Flow (MGD)** | **Basis** | **Per Capita Flow (Gal/day)** |
| **Dry Weather Flows** | | | |
| ADWF | 0.57 | Analysis of 2006-2011 DMRs (May-Oct) | 153 |
| Base Sewerage | 0.51 | Assume no I/I (July - Sept.) | 137 |
| Base Infiltration | 0.06 | ADWF - Base Sewerage | 16 |
| $MMDWF_{10}$ | 0.86 | Figure 5.1.3 (DEQ Graph No. 1) | 232 |
| **Wet Weather Flows** | | | |
| AWWF | 1.04 | Analysis of 2006-2011 DMRs (Nov.-Apr.) | 280 |
| $MMWWF_{5}$ | 1.51 | Figure 5.1.3 (DEQ Graph No. 1) | 405 |
| Peak Weak | 2.64 | Figure 5.1.4b (DEQ Graph No. 3) | 710 |
| Peak Day (PDAF) | 3.91 | Figure 5.1.4a (DEQ Graph No. 2) | 1051 |
| Peak Hourly (PIF) | 6.50 | Figure 5.1.4b (DEQ Graph No. 3) | 1747 |
| **Inflow and Infiltration** | | | |
| AWW I/I | 0.53 | AWWF -Base Sewerage | 143 |
| MMWW I/I | 1.00 | MMWWF - Base Sewerage | 269 |
| Peak Day I/I | 3.40 | PDAF - Base Sewerage | 914 |
| PI I/I | 5.99 | PIF - Base Sewerage | 1611 |

**Figure 5.1.6 - Measured Flows at Treatment Plant**



Flows calculated and summarized in Table 5.1.6 seem to correlate well with, and are validated by, the actual flow data depicted in Figure 5.1.6.

### 5.1.7. **Projected Wastewater Flows**

Projected wastewater flows are developed based on the assumption that flow per capita will hold constant. This results in the increase in projected flows being proportional to the population growth. Per Section 3.3, the population is expected to increase by nearly 16% from 2010 data to the end of the 20 year planning cycle (2032).

Projecting peak flows at the same rate of community growth results in the assumption of I/I flows increasing at a similar rate. The City is currently addressing I/I issues and has a plan in place to continue monitoring and repairing the worst areas, which will likely lead to less I/I. However, assuming a population based increase in I/I flows will lead to conservative design flows and is therefore the approach taken to flow projections.

**Table 5.1.7 Summary of Current and Projected Wastewater Flows**

| Summary of Current and Projected Wastewater Flows | | | | | | |
|---|---|---|---|---|---|---|
| **Parameter** | **2010 Flow** (MGD) | **Basis** | **2010 Population** * | **Per Capita Flow** (Gal/day) | **2032 Population** ** | **2032 Flow** (MGD) |
| **Dry Weather Flows** | | | | | | |
| ADWF | 0.57 | Analysis of 2006-2011 DMRs (May-Oct) | 3465 | 154 | 4285 | 0.66 |
| Base Sewerage | 0.51 | Assume no I/I (July - Sept.) | | 137 | | 0.59 |
| Base Infiltration | 0.06 | ADWF - Base Sewerage | | 16 | | 0.07 |
| $MMDWF_{10}$ | 0.86 | Figure 5.1.3 (DEQ Graph No. 1) | | 233 | | 1.00 |
| **Wet Weather Flows** | | | | | | |
| AWWF | 1.04 | Analysis of 2006-2011 DMRs (Nov.-Apr.) | | 282 | | 1.21 |
| $MMWWF_5$ | 1.51 | Figure 5.1.3 (DEQ Graph No. 1) | | 408 | | 1.75 |
| Peak Weak | 2.64 | Figure 5.1.4b (DEQ Graph No. 3) | | 714 | | 3.06 |
| Peak Day (PDAF) | 3.91 | Figure 5.1.4a (DEQ Graph No. 2) | | 1057 | | 4.53 |
| Peak Hourly (PIF) | 6.50 | Figure 5.1.4b (DEQ Graph No. 3) | | 1757 | | 7.53 |

* 2010 Population based 2010 census data.
** 2032 Populations per Oregon Office of Economic Analysis, Department of Administrative Services.

### 5.1.8. **Lift Stations Projected Wastewater Flows**

As each of the lift stations within the Toledo wastewater collection system were reviewed, a common concern was identified. The concern was due to the lack of flow or run time data at each of the lift stations. Current information available related to system flows is limited to the outlet flows from the wastewater treatment plant. Previous facility plans for the City's wastewater system used EDU counts and basin areas as the basis for flow determinations. This data is out of date and a new basin flow analysis was completed.

Two techniques were used to analyze Toledo's wastewater collection system. The first used 2010 Census population data for the community distributed across the existing collection basins and the PIF per capital identified in Table 5.1.7 of this report. A table for the PIF for each basin is provided:

**Table 5.1.8 - Basin PIF (Census Data)**

| Basin | Census Pop 2010 | | Adjusted Pop 2010 | PIF (g/d) |
|---|---|---|---|---|
| A | 13 | 0.003919 | 15 | 26,205 |
| B | 495 | 0.149231 | 552 | 964,344 |
| C | 196 | 0.05909 | 219 | 382,593 |
| D | 329 | 0.099186 | 367 | 641,149 |
| E | 0 | 0 | 0 | 0 |
| F | 264.5 | 0.079741 | 295 | 515,365 |
| G | 117.5 | 0.035424 | 131 | 228,857 |
| H | 104 | 0.031354 | 116 | 202,652 |
| I | 563 | 0.169732 | 628 | 1,097,116 |
| J | 0 | 0 | 0 | 0 |
| K | 297 | 0.089539 | 331 | 578,257 |
| L | 186 | 0.056075 | 207 | 361,629 |
| M | 107 | 0.032258 | 119 | 207,893 |
| N | 70 | 0.021103 | 78 | 136,266 |
| O | 299 | 0.090142 | 334 | 583,498 |
| P | 276 | 0.083208 | 308 | 538,076 |
| Total: | 3317 | 1 | 3700 | 6,463,900 |

Using the basin flows developed in Table 5.1.8.a the following peak lift station flow summary table was compiled:

**Table 5.1.8.a – Census Based Flow Analysis**

| Lift Station | Primary Basins Served | Lift Stations Served | Total Basins Served | PIF at Lift Station (g/d) |
|---|---|---|---|---|
| A Street | B, D, E, F | Hospital | B, C, D, E, F | 2.92 |
| Ammon Road | G, H, M, N, O, P | High School | A, G, H, M, N, O, P | 2.24 |
| High School | A | | | 0.03 |
| Hospital | C | | | 0.45 |
| Butler Bridge | I, J, K | A Street | B, C, D, E, F, I, J, K | 4.87 |

The second analysis of the lift station flows used the existing collection system piping as the basis for the flow determination. This investigation recognizes that the major contributor to system flows is I&I. Table 5.1.8.b summarizes the estimated total length of all of the gravity sewer lines by size within the Toledo waste water collection network and normalizes them into inch-diameter-mile based on the existing basins. This table also includes estimated service line lengths for residential, commercial, and industrial customers within the collection network.

**Table 5.1.8.b - Distribution System Summary**

| Basin Sewer Pipe Summary (Feet) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Basins | Pipe Size (Inches) | | | | | | | | Inch- |
| | 4 | 6 | 8 | 10 | 12 | 14 | 15 | 18 | Diameter- |
| A | 250 | 1,651 | | | | | | | 2.07 |
| B | 5,550 | 833 | 13,833 | | | | | | 26.11 |
| C | 2,350 | 100 | 7,932 | | | | | | 13.91 |
| D | 3,950 | 150 | 10,689 | | | | | | 19.36 |
| E | 300 | | 1,016 | | | | | | 1.77 |
| F | 6,950 | 718 | 8,221 | 593 | 804 | | 798 | 295 | 24.91 |
| G | 3,250 | 1,921 | 5,899 | | | | | | 13.58 |
| H | 870 | | 5,899 | | | | | | 9.60 |
| I | 10,450 | 996 | 13,585 | 573 | 309 | | | 1,855 | 37.74 |
| J | 100 | | 749 | | | | | | 1.21 |
| K | 4,200 | 2,120 | 5,936 | | | | | | 14.59 |
| L | 3,550 | | 4,292 | | 34 | | | | 9.27 |
| M | 2,450 | | 3,829 | 2,017 | | | | | 11.48 |
| N | 1,350 | 250 | 4,169 | 47 | 1,375 | | | | 10.84 |
| O | 4,800 | | 11,747 | | 354 | | 17 | | 22.29 |
| P | 5,150 | | 7,464 | | | | | | 15.21 |

The breakdown of the wastewater collection network provided in Table 5.1.8.b was then coupled with total system peak instantaneous flow of 6.5 mgd, identified in Table 5.1.7 to calculate total peak flow for each lift station. Table 5.1.8.c summarizes the PIF for each lift station within the Toledo wastewater collection network based on the existing collection network.

**Table 5.1.8.c - Collection System Based Flow Analysis**

| Lift Station | Primary Basins Served | Lift Stations Served | Total Basins Served | PIF at Lift Station (mgd) |
|---|---|---|---|---|
| A Street | B, D, E, F | Hospital | B, C, D, E, F | 2.39 |
| Ammon Road | G, H, M, N, O, P | High School | A, G, H, M, N, O, P | 2.36 |
| High School | A | | | 0.06 |
| Hospital | C | | | 0.39 |
| Butler Bridge | I, J, K | A Street | B, C, D, E, F, I, J, K | 3.88 |

The current calculated flows at Toledo's lift stations discussed above when compared appear to be reasonable and accurate given the information available. Due to the lack of actual flow to validate the calculated flows a short term monitoring of the WWTP headwork's flumes was completed. The existing flumes provide flow data for the Butler Bridge and Ammon Road lift stations. The data that was collect has been provided below in Table 5.1.8.d.

**Table 5.1.8.d - 2012 Actual Field Flow Data**

| Collection System Flow Monitoring | | | |
|---|---|---|---|
| Date | Bulter Bridge Lift Station (mgd) | Ammon Road Lift Station (mgd) | Rainfall (Inches) |
| 12/4/2012 | 2.281 | 0.887 | 1.44 |
| 12/5/2012 | 1.539 | 0.717 | 0.00 |
| 12/6/2012 | 1.396 | 0.592 | 0.44 |
| 12/8/2012 | 1.025 | 0.427 | 0.11 |
| 12/9/2012 | 0.924 | 0.379 | 0.18 |
| 12/10/2012 | 0.887 | 0.321 | 0.01 |
| 12/11/2012 | 0.859 | 0.308 | 0.30 |
| 12/12/2012 | 0.862 | 0.305 | 0.29 |
| 12/13/2012 | 0.779 | 0.282 | 0.05 |
| 12/14/2012 | 0.835 | 0.307 | 0.17 |
| 12/15/2012 | 0.949 | 0.324 | 0.62 |
| 12/16/2012 | 1.485 | 0.536 | 1.12 |
| 12/17/2012 | 1.227 | 0.515 | 0.20 |
| Average: | 1.16 | 0.45 | 0.38 |

The flow date in Table 5.1.8.d was then compared with the calculated flows completed above. An adjustment factor using the percentages of total flow was developed to adjust the calculated flows for each lift station within the collection system to more accurately depict the collection system flows. In Table 5.1.8.e and Table 5.1.8.f a summary of the current and projected flows within the system at each lift station is provided. When reviewing the current and future capacity of each lift station within the Toledo wastewater collection system it is recommended that the Adjusted Average Lift Station Flows provided in Tables 5.1.8.e and 5.1.8.f be used.

**Table 5.1.8.e - 2012 (Current) Weighted Lift Station Flows**

| Lift Station | Population Based PIF at Lift Station (mgd) | Collection System Based PIF at Lift Station (mgd) | Average Lift Station Flows (mgd)* | Actual Field Flow Data Adjustment | Adjusted Average Lift Station Flows (mgd) |
|---|---|---|---|---|---|
| A Street | 2.52 | 2.39 | 2.41 | 1.149 | 2.77 |
| Ammon Road | 1.93 | 2.36 | 2.30 | 0.745 | 1.75 |
| High School | 0.03 | 0.06 | 0.05 | 0.600 | 0.03 |
| Hospital | 0.38 | 0.39 | 0.39 | 1.154 | 0.45 |
| Butler Bridge | 4.20 | 3.88 | 3.93 | 1.151 | 4.51 |

* Calculated as: (15% x Population Based) + (85% x Collection Based)

**Table 5.1.8.f - Projected Weighted Lift Station Flows**

| Lift Station | Population Based PIF at Lift Station (mgd) | Collection System Based PIF at Lift Station (mgd) | Average Lift Station Flows (mgd)* | Actual Field Flow Data Adjustment | Adjusted Average Lift Station Flows (mgd) |
|---|---|---|---|---|---|
| A Street | 2.92 | 2.77 | 2.79 | 1.149 | 3.21 |
| Ammon Road | 2.24 | 2.74 | 2.66 | 0.745 | 1.98 |
| High School | 0.03 | 0.07 | 0.06 | 0.600 | 0.04 |
| Hospital | 0.45 | 0.45 | 0.45 | 1.154 | 0.52 |
| Butler Bridge | 4.87 | 4.49 | 4.55 | 1.151 | 5.24 |

* Calculated as: (15% x Population Based) + (85% x Collection Based)

Prior to establishing a formal facility improvement project at the existing lift stations within the collection system it is recommended that the City install flow meters at each of its lift stations to validate the calculated flows provided above for at least one year.

## *5.2. Wastewater Composition*

Wastewater composition refers to the solids, chemicals, organics, and other materials that make up municipal wastewater. Because wastewater is generated by residential, commercial and industrial sources, the constituents within the wastewater can vary greatly. However, the treatment requirements and treated water quality remains consistent, based upon NPDES Permit requirements.

A detailed analysis of the City of Toledo DMRs from January 2006 through June 2011 was conducted to aid in establishing a basis for long term projection of organic loading and wastewater composition for the planning period. This information will be utilized in proposing treatment processes and operations to reduce unwanted constituents in the wastewater and to ensure the City is able to meet the requirements of the NPDES discharge permit.

### 5.2.1. **Analysis of Plant Records**

Analysis of the most recent five (5) years (2006 – 2011) of Discharge Monitoring Reports (DMRs) from Wastewater Treatment Plant #2 has identified a number of parameters which characterize the City's wastewater. Plant records include influent measurement of BOD and TSS a minimum of twice per week. Figures 5.2.1a through 5.2.1.d below summarize the concentration and loading of these primary constituents.

**Figure 5.2.1a BOD Composition**

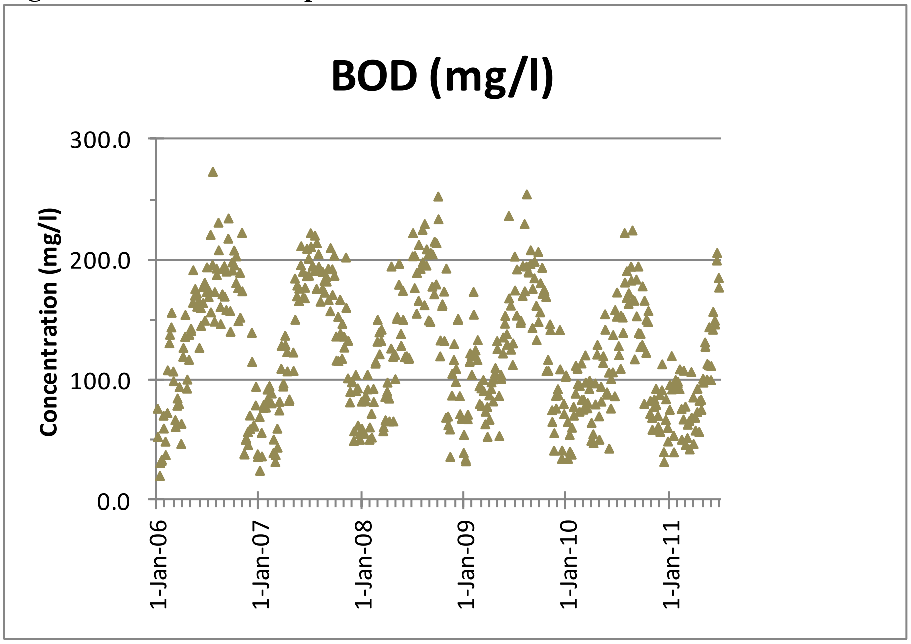



**Figure 5.2.1b BOD Influent Loading**

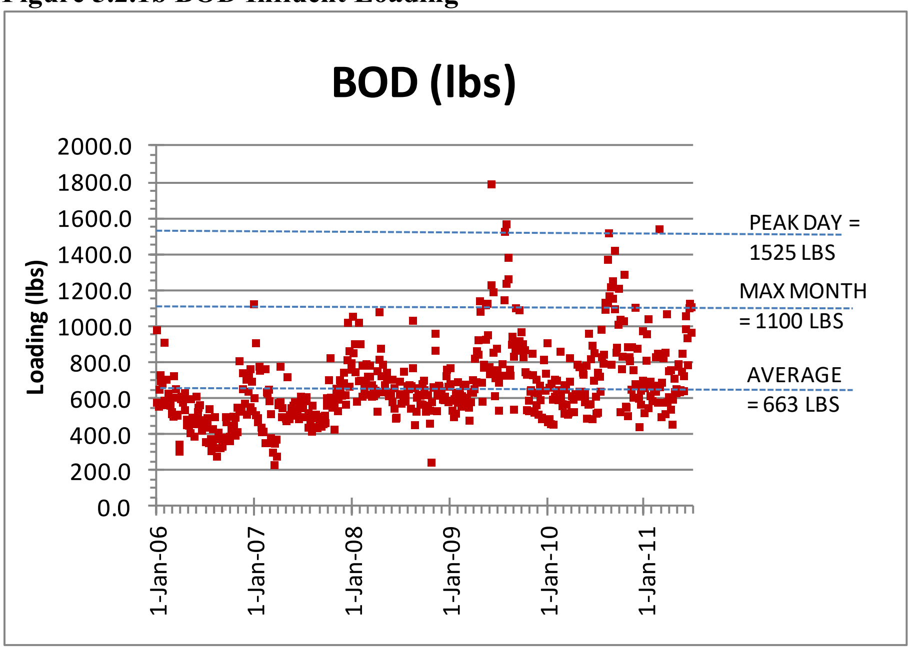


**Figure 5.2.1c TSS Composition**

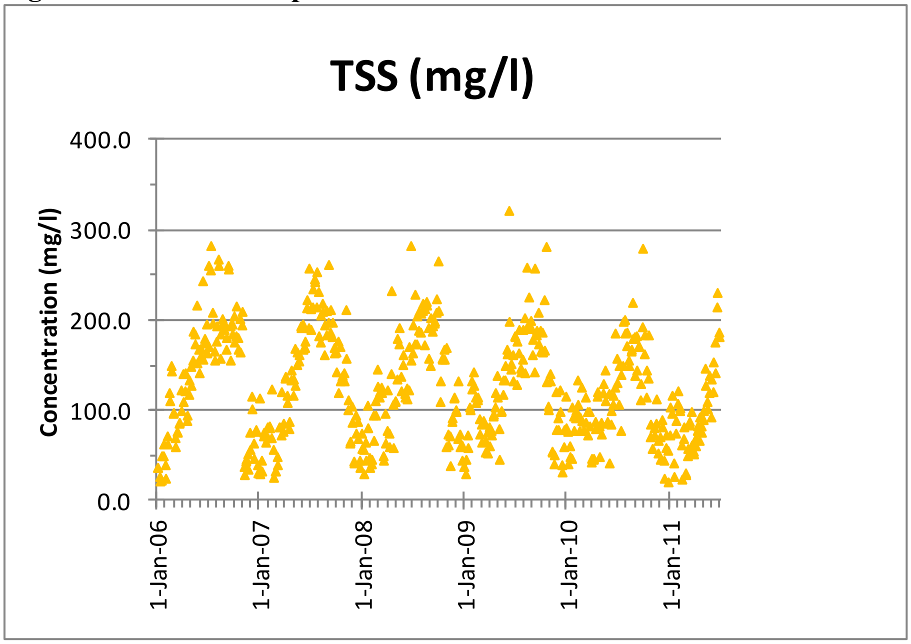
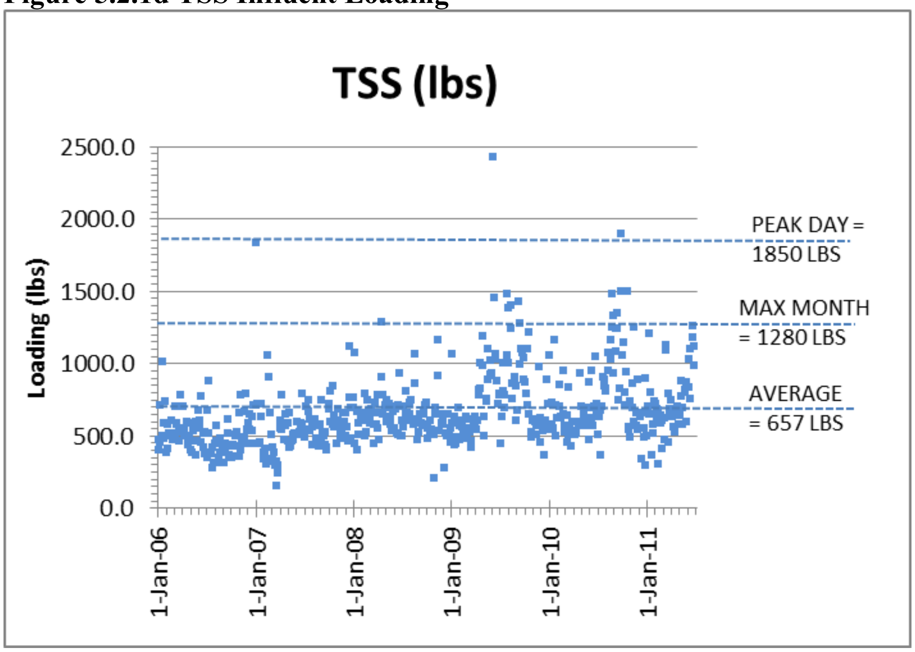



**Figure 5.2.1d TSS Influent Loading**




### 5.2.2. **Wastewater Composition**

Table 5.2.2a below identifies the composition of the influent in terms of BOD, TSS and pH.

**Table 5.2.2a Current Influent Composition**

| Current Wastewater Composition Summary | | | | | | |
|---|---|---|---|---|---|---|
| Flow Parameter | BOD | | TSS | | Ph | |
| | Composition (mg/L) | Loading (lbs) | Composition (mg/L) | Loading (lbs) | min | max |
| **Annual Average** | 123.59 | 663.47 | 124.99 | 657.54 | 6.73 | 7.28 |
| **Winter Average** | 85.33 | 653.59 | 80.91 | 614.53 | 6.56 | 7.31 |
| **Summer Average** | 163.93 | 673.89 | 170.95 | 702.39 | 6.91 | 7.49 |
| **Maximum Month** | 205.10 | 1138.11 | 227.00 | 1200.92 | 6.27 | 7.32 |
| **Maximum Day** | 250.00 | 1525 | 285 | 1850 | 5.70 | 8.20 |

As seen above, summer and winter flows had significantly different compositions of BOD and TSS, while the loading of these constituents was relatively independent of the seasonal flow fluctuations as would be expected due to the influx of I/I.

Typical concentrations of contaminants within untreated domestic wastewater are identified in the text, *Wastewater Engineering, Treatment and Reuse,* Metcalf & Eddy, 2003. Data given in the referenced text is summarized in Table 5.2.2b below for comparison to the average load concentrations shown in the table above, as measured at the Toledo WWTP.

**Table 5.2.2b Typical Composition of Untreated Domestic Wastewater**

| Typical Wastewater Composition | | | | |
|---|---|---|---|---|
| | | Concentration | | |
| Contaminant | Unit | Low Strength | Medium Strength | High Strength |
| Biochemical Oxygen Demand, 5-d, 20°C (BOD) | mg/L | 110 | 190 | 350 |
| Total Suspended Solids (TSS) | mg/L | 120 | 210 | 400 |
| Fecal Coliform | No./100mL | $10^3$-$10^5$ | $10^4$-$10^6$ | $10^5$-$10^8$ |
| Free Ammonia Nitrogen (NH3-N) | mg/L | 12 | 25 | 45 |

Source: Table 3-15, "Wastewater Engineering, Treatment and Reuse," Metcalf & Eddy, 2003.

By comparing the typical values in the above table to the overall average constituent concentrations presented in Table 5.2.2a, average influent BOD and TSS values for Toledo are considered low strength.

## *5.3. Projected Wastewater Characteristics*

As developed in section 3.3.2, the current population, as of 2010, served by the City of Toledo is 3,465 persons. Based on growth projections discussed in section 3.3, the population served at the end of the design period will be approximately 4,013 persons. Population growth is expected to occur in areas of vacant land within the city limits or within the Urban Growth Area. New collection facilities will need to be constructed in order for development to occur in many areas.

At this time, no significant change to the current ratio of residential to commercial to industrial sources is expected. Therefore, for the purposes of projecting wastewater characteristics, it is assumed that flows and loading will increase over time based on the increase in population and that the composition, per unit volume, of the wastewater will remain the same.

Projected BOD and TSS loading for Toledo in the year 2032 are summarized in Table 5.3, below, including the unit loading presented in units of pounds per person per day. The values presented have been determined by dividing the average and peak loads determined from the DMRs by the existing population to obtain unit loads (design factors) in terms of pounds per capita day. The unit design factors were then multiplied by the projected population to determine projected loading.

**Table 5.3 Summary of Current and Projected Wastewater Loads**

| Current and Projected Loading | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Parameter | 2010 Loading (lbs/day) | | 2010 Population | Unit Loading (lbs/capita/day) | | 2032 Population | 2032 Loading (lbs/day) | |
| | BOD | TSS | | BOD | TSS | | BOD | TSS |
| Annual Average | 663.47 | 657.54 | 3465 | 0.191 | 0.190 | 4285 | 820.45 | 813.12 |
| Winter Average | 653.59 | 614.53 | | 0.189 | 0.177 | | 808.24 | 759.93 |
| Summer Average | 673.89 | 702.39 | | 0.194 | 0.203 | | 833.34 | 868.58 |
| Maximum Month | 1138.11 | 1200.92 | | 0.328 | 0.347 | | 1407.39 | 1485.06 |
| Maximum Day | 1525.00 | 1850.00 | | 0.440 | 0.534 | | 1885.83 | 2287.72 |

# 6.0 Basis of Planning

Section

6

## 6.1. Basis for Design

### 6.1.1. Regulatory Requirements

The Clean Water Act (CWA) as delegated to the State of Oregon and enforced through Oregon Revised Statues (ORS 468B.050), requires permits for all discharges of wastewater to waters of the state. The City of Toledo operates its wastewater system under the jurisdiction of the Oregon Department of Environmental Quality (DEQ), with a National Pollutant Discharge Elimination System (NPDES) Waste Discharge Permit (Permit No. 101713) which was issued on December 27, 2005 (See Appendix A). NPDES permits are generally issued for terms of 5 years, at which time any changes to the rules will be included in the renewed permit. When a facility's permit reaches the expiration date and a new permit is not issued, the current permit is administratively extended and the permit requirements remain in effect provided that the permittee has made timely application for renewal. An NPDES Permit application was submitted to DEQ in June of 2010, the City of Toledo has not yet received a new NPDES Permit. Based on discussions with DEQ, it was unlikely that a new permit would be issued until the next permit cycle (2015).

The 2005 NPDES permit allows the City to discharge treated wastewater to the Yaquina River at river mile 13.7 under the prescribed effluent limitations and other requirements. These effluent limits are developed to protect the beneficial uses for the Mid Coastal Basin (Oregon Administrative Rules 340-041-0220).

Oregon Administrative Rules (OAR) also contain both statewide and basin specific minimum design criteria and rules regarding sanitary sewage overflows. These rules are discussed below:

#### 6.1.1.1. Minimum Design Criteria for Wastewater Treatment and Control of Wastes

OAR 340-041-0007 (Statewide Narrative Criteria) includes minimum design criteria for treatment and control of wastes. Generally, wastewater from a municipal wastewater treatment system must be treated and controlled in facilities designed in accordance with the following minimum criteria:

- In designing treatment facilities, average conditions and a normal range of variability are generally used in establishing design criteria. A facility once completed and placed in operation should operate at or near the design limit most of the time but may operate below the design criteria limit at times due to variables which are unpredictable or uncontrollable. This is particularly true for biological treatment facilities. The actual operating limits are intended to be established by permit pursuant to ORS 468.740 and recognize that the actual performance level may at times be less than the design criteria.
- Effluent BOD concentrations in mg/l, divided by the dilution factor (ratio of receiving stream flow to effluent flow) may not exceed one unless otherwise approved by the Commission;
- Sewage wastes must be disinfected, after treatment, equivalent to thorough mixing with sufficient chlorine to provide a residual of at least 1 part per million after 60 minutes of contact time unless otherwise specifically authorized by permit;
- Positive protection must be provided to prevent bypassing raw or inadequately treated sewage to public waters unless otherwise approved by the Department where elimination of inflow and infiltration would be necessary but not presently practicable; and

- More stringent waste treatment and control requirements may be imposed where special conditions make such action appropriate.

OAR 340-041-0225 (Water Quality Standards and Policies for the Mid Coast Basin) includes minimum design criteria for treatment and control of wastes. These are as follows:

- pH values by not fall outside the range of 6.5 to 8.5.
- During periods of low stream flows (approximately May 1 to October 31), treatment resulting in monthly average effluent concentrations not to exceed 20 mg/l of BOD and 20 mg/l of SS or equivalent control;
- During the period of high stream flows (approximately November 1 to April 30), a minimum of secondary treatment or equivalent control and unless otherwise specifically authorized by the Department, operation of all waste treatment and control facilities at maximum practicable efficiency and effectiveness so as to minimize waste discharges to public waters.

New or expanded wastewater systems must meet the requirements described above.

#### 6.1.1.2. Sanitary Sewage Overflows (SSOs)

OAR 340-041-0009 (6) and (7) prohibit discharging of raw sewage to wastewaters of the state in the winter and summer, respectively. During the summer (May 22 through October 31), raw sewage discharges are prohibited, except during a storm event greater than the one-in-ten year 24-hour duration storm. Since January 1, 2010, raw sewage discharges are prohibited during the winter (November 1 through May 21), except during a storm event greater than the one-in-five year, 24-hour duration storm.

### 6.1.2. **Water Quality Status of Receiving Waterbody**

Per OAR 340-041-0004, the Antidegradation Policy guides decisions that affect water quality such that unnecessary further degradation from new or increased point and nonpoint sources of pollution is prevented, and enhances existing surface water quality to ensure the full protection of all existing beneficial uses.

#### 6.1.2.1. Clean Water Act, Section 303(d)

Section 305(b) of the Clean Water Act (CWA) requires DEQ to assess water quality in Oregon and report on the overall condition of waters. DEQ assigns an assessment status category to each water body where data are available to evaluate. Water bodies that do not meet water quality standards are Water Quality Limited and are assigned Category 4 or Category 5. Water bodies in Category 5 need pollutant Total Maximum Daily Loads (TMDLs) developed and comprise the Section 303(d) list.

Table 6.1.2.1 below summarizes the water quality status of the Yaquina River near the City of Toledo.

**Table 6.1.2.1 Yaquina River Water Quality Status**

| Parameter | Season | Criteria | Status | Year | Action |
|---|---|---|---|---|---|
| Alkalinity | Year Around | Table 20 Toxic Substances | Cat 3B: Potential concern | 2004 | No 2010 action |
| Ammonia | Year Around | Table 20 Toxic Substances | Cat 2: Attaining some criteria/uses | 2004 | No 2010 action |
| Barium | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |
| Beryllium | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |
| Cadmium | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |
| Chloride | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |
| Chromium (hex) | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |
| Copper | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |
| Dissolved Oxygen | Year Around (Non-spawning) | Cold water: Not less than 8.0 mg/l or 90% of saturation | Cat 5: Water quality limited, 303(d) list, TMDL needed | 2004 | No 2010 action |
| Fecal Coliform | Year Around | Fecal coliform median of 14 organisms per 100 ml; no more than 10% > 43 organisms per 100 ml | Cat 5: Water quality limited, 303(d) list, TMDL needed | 1998 | No 2010 action |
| Iron | Year Around | Table 20 Toxic Substances | Cat 3B: Potential concern | 2004 | No 2010 action |
| Manganese | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |
| Nickel | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |
| Phosphate Phosphorus | Summer | Total phosphates as phosphorus (P): Benchmark 50 ug/L in streams to control excessive aquatic growths | Cat 2: Attaining some criteria/uses | 2004 | No 2010 action |
| Silver | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |
| Zinc | Year Around | Table 20 Toxic Substances | Cat 3: Insufficient data | 2004 | No 2010 action |

In the area of the discharge (River Mile 13.7) the Yaquina River is Water Quality Limited, 303(d) list, for Dissolved Oxygen (2004) and Fecal Coliform (1998) per the Oregon 2010 Integrated Report.

### 6.1.2.2. Temperature

Water temperatures affect the biological cycles of aquatic species and are a critical factor in maintaining and restoring healthy salmonid populations throughout the state. It is the policy of the Environmental Quality Commission (EQC) to protect aquatic ecosystems from adverse warming caused by anthropogenic activities. The purpose of the temperature criteria listed in OAE 340-041-0028 is to protect designated temperature sensitive beneficial uses, including salmonid life cycle stages in waters of the State.

DEQ’s Fish Use Designation maps identify the applicable temperature criteria for each basin. The mid Coast sub-basin map is set out in 340-041-0220A and -0220B. According to the Fish Use Designation maps approved with the temperature standard, the Yaquina River in this area is designated as a rearing and migration corridor.

The DEQ list of Water Quality Limited Water Bodies for 2010 indicates that the Yaquina is not water quality limited for temperature during the summer in the area of the outfall. However, in order to protect cold water, a point source may not increase the stream temperature (at the point of maximum impact) by more than 0.3 degrees Celsius above the ambient temperature (OAR 340-041-0028(11)(a)).

Based on the existing discharge (existing facility design flow and maximum effluent temperature), DEQ calculated the in-stream temperature increases based on the dilution achieved in the mixing zone. A dilution of 13:1 was calculated at the edge of the mixing zone. DEQ's ambient data collection from the Yaquina River shows that 16° C is the 90$^{th}$ percentile for the lowest background temperature in the Yaquina River near the outfall. A temperature of 23° C was calculated as the 90$^{th}$ percentile effluent temperature.

Because the in-stream temperature increase is larger than the allowable increase, DEQ has determined that the facility has a reasonable potential to violate the temperature standard. Therefore, an Excess Thermal Load (ETL) limit was placed in the current permit. The ETL is based on dilution achieved in the mixing zone because that is the most stringent limit. The current limit is 11 million kcals per day as a weekly average and is likely to remain on the upcoming permit renewal.

#### 6.1.2.3. Total Chlorine Residual

Disinfection of the effluent with chlorine is the process the plant is designed to use in order to comply with the waste discharge limitations for bacteria. Chlorine is a known toxic substance and as such is subject to limitation under Oregon Administrative Rules. The rule (OAR 340-041-0033(2)) states, in part, that toxic substances shall not be discharged to waters of the state at levels that adversely affect public health, aquatic life or other designated beneficial uses. In addition, levels of toxic substances shall not exceed the criteria listed in Table 20 which were based on criteria established by the EPA and published in Quality Criteria for Water (1986), unless otherwise noted.

However, OAR 340-041-0053(2)(b)(A) states that the DEQ may allow a designated portion of a receiving water to serve as a zone of dilution for wastewaters and receiving waters to mix thoroughly and this zone will be defined as a mixing zone. DEQ may suspend all or part of the water quality standards, or set less restrictive standards, in the defined mixing zone, provided the water within the mixing zone is free of materials in concentrations that will cause acute toxicity to aquatic life as measured by the acute bioassay method and outside the boundary of the mixing zone is free of materials in concentrations that will cause chronic toxicity.

Furthermore, 40 CFR §122.44(d) states that permit limitations must control all pollutants or pollutant parameters which are, or may be, discharged at a level which will cause, have the reasonable potential to cause, or contribute to an excursion above any state water quality standard, including state narrative criteria for water quality. According to OAR 340-041, Table 20, chlorine concentrations of 11µg/L can result in chronic toxicity in fresh waters while 19 µg/L can result in acute chlorine toxicity in fresh waters.

Dilutions at the edge of the mixing zone and at the zone of immediate dilution (based on the *Yaquina River Mixing Zone Modeling Study for City of Toledo, Oregon,* prepared by Scott A. Wells, Ph.D., P.E.), effluent data for chlorine residual, and the average dry weather and wet weather design flows for the facility were entered into a DEQ Reasonable Potential Analysis (RPA) spreadsheet program to determine whether there is a reasonable potential to violate the instream water quality standards for chlorine at the edge of the mixing zone and zone of immediate dilution (ZID). The RPA indicated there was a reasonable potential to violate the chlorine standard year round.

Because there is a reasonable potential to violate the chlorine toxicity standard year round, permit limitations based on the dilution provided in the river at the worst case scenario for acute and chronic criteria for winter and summer were added to the 2005 permit and are likely to remain in effect for the upcoming permit renewal. 2005 permitted discharge parameters stated that the Total Chlorine Residual "Shall not exceed 0.01 mg/L monthly average and 0.02 mg/l daily maximum."

#### 6.1.2.4. Ammonia

Ammonia is a substance normally found in wastewater. The wastewater treatment processes, particularly aeration and biological treatment, can convert a large portion to nitrate and nitrite but the treated effluent still contains some ammonia. After discharge, the continued process of oxidizing the ammonia removes dissolved oxygen from the ambient water.

Unionized ammonia is also a toxic agent and may have to be limited to prevent toxicity. As with chlorine residual, the water outside the boundary of the mixing zone shall be free of materials in concentrations that will cause chronic (sub-lethal) toxicity while the water outside the ZID must be free of pollutants that will cause acute toxicity. If ammonia is discharged at a level which will cause, have the reasonable potential to cause, or contribute to an excursion above any state water quality standard (dissolved oxygen or toxicity), it must be limited by the permit.

The NPDES Permit Evaluation Report, August 23, 2004, which was prepared prior to the issuance of the current permit determined that there was no reasonable potential to violate either the chronic or acute toxicity standard. However, since the report was prepared dissolved oxygen was added to the 303(d) list. Because of this, DEQ will likely be concerned regarding ammonia discharges into the Yaquina River. As part of the permit renewal application process, DEQ asked for the City of Toledo to submit a minimum of ten ammonia sample results. Out of the 11 ammonia results provided to DEQ, 3 were at level below the detectable limit, 7 were between 1.1 and 1.8 mg/L, and one was 130 mg/L. Levels at non-detect or within the range of the other 7 samples do not pose any concern, however the 130 mg/L sample is concerning. The 130 mg/L sample was taken on May 10, 2010 and only five days earlier a sample showed non-detect and a sample taken three days later showed 1.1 mg/L. Due to this large discrepancy, it seems likely that there was an error in the sampling or testing procedure. To double check this assertion, we checked the plant records for that day and found that their internal sampling resulted in an effluent ammonia concentration of 0.24 mg/l.

Because the historical ammonia discharge is so small, it is unlikely that there will be any ammonia limits added to the permit during the course of the planning period.

### 6.1.3. **Effluent Quality**

Based on the discussions is section 6.1.2 above, changes to the existing permit limitations are not expected. Therefore, the planned permit limitations are the same as current permit limitations as described in Schedule A of the current permit summarized below.

**Table 6.1.3 - NPDES Permit Schedule A - Waste Discharge Limitations not to be exceeded**

(1) May 1 – October 31:

| Parameter | Average Effluent Concentrations Monthly | Average Effluent Concentrations Weekly | Monthly* Average lb/day | Weekly* Average lb/day | Daily* Maximum lbs |
|---|---|---|---|---|---|
| $BOD_5$ | 10 mg/L | 15 mg/L | 61 | 91 | 120 |
| TSS | 10 mg/L | 15 mg/L | 61 | 91 | 120 |

(2) November 1 – April 30:

| Parameter | Average Effluent Concentrations Monthly | Average Effluent Concentrations Weekly | Monthly* Average lb/day | Weekly* Average lb/day | Daily* Maximum lbs |
|---|---|---|---|---|---|
| $BOD_5$ | 20 mg/L | 30 mg/L | 270 | 410 | 550 |
| TSS | 20 mg/L | 30 mg/L | 270 | 410 | 550 |

- Average dry weather design flow to the facility equals 0.73 MGD. Summer mass load limits based upon average dry weather design flow to the facility. Winter mass load limits based upon average wet weather design flow to the facility equaling 1.64 MGD. The daily mass load limit is suspended on any day in which the flow to the treatment facility exceeds 1.46 MGD (twice the design average dry weather flow)

(3)

| Other parameters (year-round) | Limitations |
|---|---|
| Fecal Coliform Bacteria | Shall not exceed a 40 day log mean of 100 organisms per 100 mL and a weekly log mean of 200 organisms per 100 mL. (See Note 1) |
| pH | Shall be within the range of 6.0 – 9.0 |
| $BOD_5$ and TSS Removal Efficiency | Shall not be less than 85% monthly average |
| Total Chlorine Residual | Shall not exceed 0.01 mg/l monthly average and 0.02 mg/l daily maximum (See Notes 2 and 3) |
| Excess Thermal Load (ETL) | Shall not exceed a weekly average of 11 million Kcals/day (See Note 4) |

(4)

Except as provided for in OAR 340-045-0080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-041-0245 except in the following defined mixing zone:

The allowable mixing zone is that portion of the Yaquina River extending out one hundred (100) feet from the east bank of the river and extending from a point one hundred (100) feet upstream of the outfall to a point one hundred (100) feet downstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within ten (10) feet of the point of discharge.

NOTES:

1. At the point of discharge, the Yaquina River is water quality limited for bacteria year-round. A

Total Maximum Daily Load (TMDL) has not been issued for these parameters at the time of permit issuance. Upon EPA approval of a TMDL addressing this pollutant, this permit may be reopened to include any Waste Load Allocation (WLA), best management practice or any other condition required by the TMDL.

2. When the total residual chlorine limitation is lower than 0.10 mg/L, the Department will use 0.10 mg/L as the compliance evaluation level (i.e. daily maximum concentrations below 0.10 mg/L will be considered in compliance with the limitations).
3. ~~The total chlorine residual limitations shall not apply until completion of the compliance schedule in Schedule C Condition 3, or no later than the expiration date of this permit, whichever is sooner.~~ *(Chlorine residual limitation went into effect in 2009)*
4. The thermal load limit was calculated using the average dry weather design flow and an estimated maximum weekly effluent temperature. The Excess Thermal Load limit is considered interim and may be adjusted up or down or eliminated when more accurate effluent temperature data becomes available. In addition, upon approval of a Total Maximum Daily Load for temperature for this sub-basin, this permit may be re-opened to include new or revised limits or other conditions or requirements regarding temperature and/or thermal loads.

### 6.1.4. **Treatment Effectiveness**

A minimum level of percent removal for $BOD_5$ and TSS for municipal dischargers is required by the Code of Federal Regulations (CFR) secondary treatment standards (40 CFR, Part 133). An 85 percent removal efficiency limit is included in the permit to comply with federal requirements. Evaluation of the past DMRs shows that the standard removal efficiency is 96.7% for BOD and 96.6% for TSS.

### 6.1.5. **System Reliability and Redundancy Requirements**

New or expanding wastewater treatment plants should be designed to meet minimum reliability standards as described in EPA's technical bulletin, Design Criteria for Mechanical, Electric, and Fluid System and Component Reliability, EPA 430-99-74-001, 1974. These standards shall be achieved in order to ensure effective operation of treatment facilities on a day-to-day basis as well as during emergencies including power failures, flooding, peak flows, and equipment failures. These reliability standards are critical to protect the receiving water body against degradation during maintenance shutdowns and emergencies.

The above referenced EPA technical bulletin identifies the following three reliability classes:

> Reliability Class I – Works which discharge into navigable waters that could be permanently or unacceptably damaged by effluent which was degraded in quality for only a few hours. Examples of Reliability Class I works might be those discharging near drinking water reservoirs, into shellfish waters, or in close proximity to areas used for water contact sports.
>
> Reliability Class II – Works which discharge into navigable waters that would not be permanently or unacceptably damaged by short-term effluent quality degradations, but could be damaged by continued (on the order of several days) effluent quality degradation. An example of a Reliability Class II works might be one which discharges into recreational waters.
>
> Reliability Class III – These are works not otherwise classified as Reliability Class I or Class II.

The beneficial uses of the Mid Coast Basin are industrial water supply, fish and aquatic life (including salmonid passage), wildlife and hunting, fishing, boating, water contact recreation, aesthetic quality, and commercial navigation and transportation. Since the Yaquina River is a shellfish growing area, a fishing and hunting area, and is sometimes used for water contact sports, Class I reliability is required.

Lift stations shall be designed to pass the peak hydraulic flow with the largest pump out of service and major wastewater treatment process components will be designed to pass the peak wet weather flow without overflowing. The WWTP will be designed to meet all permit conditions during the maximum month dry weather flow with full redundancy of the major processes. Mechanical components in the facility will be designed to enable repair or replacement without violating the effluent limits.

The following table provides a summary of component redundancy requirements for the City Toledo wastewater treatment facilities, which include the pump stations and the treatment plant:

**Table 6.1.5 - Reliability Class I Process Requirements**

| Unit Process | Design Basis | Current Flows (MGD) | 2033 Flows (MGD) | Minimum Required Conditions |
|---|---|---|---|---|
| Influent Pumping | PIF | 6.50 | 7.53 | Firm capacity with largest pump out of service. |
| Influent Screening | PIF | 6.50 | 7.53 | Mechanically cleaned primary screen sized for PIF. Manually cleaned bar rack backup screen sized for PIF. |
| Grit Removal | MMWWF | 1.51 | 1.75 | If required for subsequent treatmetn processes, minimum of two units, each designed for peak flow (PIF). If not, a single unit is acceptable for MMWWF. |
| Aeration Basin and Clarifier | PIF | 6.50 | 7.53 | Must provide hydraulic capacity for PIF or one hour of storage capacity at PIF. |
| | PDF | 3.91 | 4.53 | Must be able to meet daily maximum discharge limits under PDF condition with both basins online. |
| | MMDWF | 0.86 | 1.00 | Must be able to meet monthly average discharge limits at MMDWF with largest basin off line. |
| Aeration Blowers | | | | Must be able to supply the design air capacity with the largest blower out of service. Minimum of two blowers required. |
| Air Diffusers | | | | Must be able to isolate and turn off largest section of diffusers within a basin without impairing oxygen transfer. |
| Disinfection | PIF | 6.50 | 7.53 | Peak flow with full redundancy. Chlorination systems must be able to meed peak demand conditions with largest feed pump out of service. Minimum two feed pumps required for chlorine service. |
| Chlorine Contact Chamber | PIF | 6.50 | 7.53 | Sufficient volume to provide 15 minutes contact time. |
| | MMWWF | 1.51 | 1.75 | Sufficient volume to provide 30 minutes contact time. |
| | MMDWF | 0.86 | 1.00 | Sufficient volume to provide 60 minutes contact time. |
| Outfall | PIF | 6.50 | 7.53 | Must be able to convey PIF under worst case hydraulic conditions (100 year flood elevation/High High Tide) |
| Electrical Power | PIF | 6.50 | 7.53 | Two separate and independent sources of electrical power are required. Primary power from utility service provider, back-up power from on-site generator. Back-up generator must have sufficient capacity to operate all vital process components, critical lighting and necessary ventilation during PIF conditions. |

### 6.1.6. Design Concepts and Constraints

The City of Toledo Wastewater Treatment Plant and the individual pump stations are all on property owned by the City. Each of the properties is relatively dense with wastewater works and does not leave much room for significant expansion. Alternatives reviewed herein take this into account and, as much as possible, remain within the existing footprints.

## 6.2. Basis for Cost Estimate

The cost estimates presented in this report will typically include four components: construction cost, engineering cost, contingency, and legal and administrative costs. Each of the cost components is discussed in this section. The estimates presented herein are preliminary and are based on the level and detail of planning presented in this Study. The goal of these planning level cost estimates is to establish a reasonably conservative budget and to allow fair cost-comparisons of alternatives. As projects proceed and more detailed, site-specific information becomes available, the estimates will require updating.

### 6.2.1. Construction Costs

Construction costs are based on competitive bidding as public works projects with Davis-Bacon prevailing wage rates. The estimated construction costs in this report are based on actual construction bidding results from similar work, published cost guides, budget quotes obtained from equipment suppliers, and other construction cost experience. Construction costs are preliminary budget level estimates prepared without design plans and details.

Future changes in the cost of labor, equipment, and materials may justify comparable changes in the cost estimates presented herein. For this reason, common engineering practices usually tie the cost estimates to a particular index that varies in proportion to long-term changes in the national economy. The Engineering News Record (ENR) construction cost index (CCI) is most commonly used. This index is based on the value of 100 for the year 1913. Average yearly values for the past 13 years are summarized in Table 6.4.1-1.

**Table 6.2.1 ENR Construction Cost Index History**

| YEAR | INDEX | % CHANGE/YR |
|---|---|---|
| 2000 | 6221 | 2.67 |
| 2001 | 6343 | 1.96 |
| 2002 | 6538 | 3.07 |
| 2003 | 6694 | 2.39 |
| 2004 | 7115 | 6.29 |
| 2005 | 7446 | 4.65 |
| 2006 | 7751 | 4.10 |
| 2007 | 7967 | 2.78 |
| 2008 | 8310 | 4.31 |
| 2009 | 8570 | 3.13 |
| 2010 | 8801 | 2.69 |
| 2011 | 9070 | 3.06 |
| 2012 | 9309 | 2.64 |
| | Average since 2000 | 3.4% |

Cost estimates presented in this report are based on average 2012 dollars with an ENR CCI of 9309. For construction performed in later years, estimated costs should be projected based on the then current year ENR Index using the following method:

Updated Cost = Report Cost Estimate x (current ENR CCI / 9309)

### 6.2.2. **Contingencies**

A contingency factor equal to approximately twenty percent (20%) of the estimated construction cost has been added to the budgetary costs estimated in this report. In recognition that the cost estimates presented are based on conceptual planning, allowances must be made for variations in final quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigation and studies, and other difficulties which cannot be foreseen at this time but may tend to increase final costs. Upon final design completion of any project, the contingency can be reduced to 10%. A contingency of at least 10% should always be maintained going into a construction project to allow for variances in quantities of materials and unforeseen conditions.

### 6.2.3. **Engineering**

Engineering services for major projects typically include surveying, preliminary and final design, preparation of contract/construction drawings and specifications, bidding services, construction management, inspection, construction staking, start-up services, and the preparation of operation and maintenance manuals. Depending on the size and type of project, engineering costs may range from 18 to 25% of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small or complicated projects.

Engineering costs for basic design and construction services presented in this report are estimated at 20% of the estimated total construction cost. Other engineering costs such as specialized geotechnical explorations, hydro-geologic studies, easement research and preparation, pre-design reports, and other services outside the normal basic services will typically be in addition to the basic engineering fees charged by firms. When it was suspected that a specific project in this report may need any special engineering services, an effort has been made to include additional budget costs for such needs. Specific efforts required for individual basic engineering tasks such as surveying, design, construction management, etc. vary widely depending on the type of project, scheduling and timeframes, level of service desired during construction, and other project/site-specific conditions however an approximate breakdown of the 20% engineering budget is as follows:

Surveying and Data Collection – 0.5%
Civil/Mechanical Design – 8%
Electrical/Controls Design – 1.5%
Bid Phase Services – 1%
Construction Management – 4%
Construction Observation (Inspection) – 5%

### 6.2.4. **Legal and Management**

An allowance of five percent (5%) of construction cost has been added for legal and other project management services. This allowance is intended to include internal project planning and budgeting, funding program management, interest on interim loan financing, legal review fees, advertising costs, wage rate monitoring, and other related expenses associated with the project that could be incurred.

### 6.2.5. **Land Acquisition**

Some projects may require the acquisition of additional right-of-way, property, or easements for construction of a specific improvement. The need and cost for such expenditures is difficult to predict and must be reviewed as a project is developed. Effort was made to include costs for land acquisition, where expected, within the cost estimates included in this report.

## 6.3. Water Balance Analysis of Wastewater Treatment Impoundments

As discussed in section 4.3.2, the excess of peak flows surpassing the treatment capacity of TU1 and TU2 are routed into the old TU1 clarifier which has a capacity of 190,000 gallons. To determine the viability of using this as a buffer, a water balance must be run. It can be assumed that the PIF will last one hour and for this calculation that the surge tank starts empty.

For the water balance, the flows into the surge tank will be the difference in the combined (TU1 + TU2 = 4.8MGD, 3331 gpm) treatment capacity and the incoming flow. For the 20 year design PIF (7.53 MGD, 5225 gpm) that equates to a storage requirement of approximately 1900 gallons per minute. Assuming the event lasts for 60 minutes, there is a required storage volume of 114,000 gallons. Since the storage capacity of the surge tank is 190,000, there is sufficient volume to buffer the PIF.

After the peak event, when flows lessen, the surge tank return pump begins to empty the basin back into the flow control structure. The surge tank return pump can pump up to 694 gpm. At that rate, it is able to return the bypassed flow into the treatment train within approximately 160 minutes.

## 6.4. Design Capacity of Conveyance System and Wastewater Treatment Plant

### 6.4.1. Conveyance System

The conveyance system must be designed to convey the Peak Instantaneous Flow (PIF).

### 6.4.2. Wastewater Treatment Plant Facilities

See figure 6.4.2 on the following page for a process by process description of the design capacity versus the Class 1 process requirements.

### 6.4.3. Seasonal Land Irrigation

The City land applies thickened sludge which meets class B biosolids requirements during the summer months. During the summer of 2012, the City land applied 258,000 gallons of solids at an average of . 3.32% solids.

**Figure 6.4.2 –Design Capacity of Wastewater Treatment Plant Facilities**

| Unit Process | Design Basis | Current Flows (MGD) | 2033 Flows (MGD) | Minimum Required Conditions | Existing Facilities Condition | Existing Facilities Meet Class 1 Process Requirements? |
|---|---|---|---|---|---|---|
| Influent Pumping | PIF | 6.50 | 7.53 | Firm capacity with largest pump out of service. | The current calculated PIF at the Butler Bridge Lift Station and the Ammon Road Lift Station is 3.92 and 2.35 MGD respectively. The anticipated design year PIFs are 4.55 and 2.66 MGD respectively. The current firm capacities of the lift stations are 3.11 and 1.81 MGD respectively. | NO |
| Influent Screening | PIF | 6.50 | 7.53 | Mechanically cleaned primary screen sized for PIF. Manually cleaned bar rack backup screen sized for PIF. | Hydraulic capacity of mechanically cleaned shaftless auger screen is 4.3 MGD. Capacity of Manually claned backup rack screen is 4.3 MGD. | NO |
| Grit Removal | MMWWF | 1.51 | 1.75 | If required for subsequent treatmetn processes, minimum of two units, each designed for peak flow (PIF). If not, a single unit is acceptable for MMWWF. | The capacity of the existing Pista grit chamber is 6.6 MGD, well above the required 1.75 MGD. | YES |
| Aeration Basin and Clarifier | PIF | 6.50 | 7.53 | Must provide hydraulic capacity for PIF or one hour of storage capacity at PIF. | Each secondary treatment unit (TU1 and TU2) are hydraulically capable of passing both the current and projected PIF. | YES |
| | PDF | 3.91 | 4.53 | Must be able to meet daily maximum discharge limits under PDF condition with both basins online. | TU1 and TU2 have design capacities of 1.5 and 2.8 MGD respectively. Combined (4.3 MGD) this is adequate for the current flows. When used in conjunction with the 0.19 MG Surge Tank, the capacity of the secondary treatment units meet Class 1 process requirements for both current and projected flows. | YES |
| | MMDWF | 0.86 | 1.00 | Must be able to meet monthly average discharge limits at MMDWF with largest basin off line. | The smaller treatment unit is capable of treating 1.5 MGD. | YES |
| Aeration Blowers | | | | Must be able to supply the design air capacity with the largest blower out of service. Minimum of two blowers required. | The design air capacity for both the aeration basins and the digesters is 1,896 scfm. The firm capacity of the existing blowers is 2590 scfm. | YES |
| Air Diffusers | | | | Must be able to isolate and turn off largest section of diffusers within a basin without impairing oxygen transfer. | It appears that each aeration basin (TU1 & TU2) has sufficient valving. | YES |
| Disinfection | PIF | 6.50 | 7.53 | Peak flow with full redundancy. Chlorination systems must be able to meed peak demand conditions with largest feed pump out of service. Minimum two feed pumps required for chlorine service. | The chlorine injection pump is capable of injecting 95 gpd. The de-chlor pump is the same pump, capable of injecting 95 gpd of 25% sodium bisulfate. There is one backup pump which can be used for either. | YES |
| Chlorine Contact Chamber | PIF | 6.50 | 7.53 | Sufficient volume to provide 15 minutes contact time. | Chlorine Contact chambers (FC1 and FC2) have a combined volume of 128,000 gallons. At the PIF, this provides 28 and 24 minutes of contact time in the current and projected flows. | YES |
| | MMWWF | 1.51 | 1.75 | Sufficient volume to provide 30 minutes contact time. | At MMWWF, the current chlorine contact chambers provide 120 and 105 minutes of contact time. | YES |
| | MMDWF | 0.86 | 1.00 | Sufficient volume to provide 60 minutes contact time. | At MMDWF, the current chlorine contact chambers provide 214 and 184 minutes of contact time. | YES |
| Outfall | PIF | 6.50 | 7.53 | Must be able to convey PIF under worst case hydraulic conditions (100 year flood elevation/High High Tide) | During peak flows the operators have noted that the hydraulic capacity of the outfall is insufficient to pass the flow during high tides. | NO |
| Electrical Power | PIF | 6.50 | 7.53 | Two separate and independent sources of electrical power are required. Primary power from utility service provider, back-up power from on-site generator. Back-up generator must have sufficient capacity to operate all vital process components, critical lighting and necessary ventilation during PIF conditions. | The current plant gets the primary power from Central Lincoln PUD. Backup power comes from a 250 kW diesel generator. | YES |

Section
7

# 7.0 Development and Evaluation of Alternatives

Section 7 will identify various alternatives for each sector and component of the wastewater system. When appropriate, cost estimates will be provided for specific alternative improvements. Also, when appropriate, a discussion will be provided to outline the advantages and disadvantages of the various alternatives. Finally, a recommendation will be provided as to which alternative is most appropriate.

The planning pattern described above will be used to analyze and develop recommendations for the conveyance system (collection and pumping systems) as well as individual components at the treatment plant. Detailed costs will be utilized to develop and present the final recommendations for sewerage system improvements in Toledo.

## *7.1. Conveyance System Alternatives*

The City of Toledo owns and maintains a wastewater conveyance system for the collection and transmission of municipal wastewater. As identified in Chapter 4, the conveyance system is composed of gravity sewer piping and manholes, as well as five wastewater lift stations and their associated force mains. Furthermore, the conveyance system has been divided into fifteen sewer basins. An Existing Conveyance System Map is presented in Figure 4.1.

The following subsections will investigate various alternatives for improvements to wastewater lift stations, collection system improvements and alternatives to consider for servicing areas within the UGB that are currently not serviced.

### 7.1.1. **Collection System Improvements and Alternatives**

The City has been working on collection system improvements and I/I reduction for well over a decade. As a result, very few new collection system piping projects need to be independently identified and discussed as a part of this Facilities Planning effort. In 2009 the City commissioned a system wide I & I study to be completed. This study was conducted by Civil West Engineering Services, Inc. The study was finalized in 2011; a copy of the study can be found in Appendix C. A brief summary of the results from the I/I survey, recommended system repairs, and the capital improvement plan defined in the I/I study are provided below.

#### 7.1.1.1. I/I Study Summary

Three investigative surveys were provided by Civil West to pinpoint I/I sources within the system. Smoke testing discovered nearly 200 individual deficiencies in the collection system, flow mapping discovered 8 large pipe and 17 manhole deficiencies, and television inspection discovered dozens of mainline pipe and lateral deficiencies.

Analysis of the surveys during this I/I report facilitated the creation of many individual improvement projects. In summary those projects consist of:

- 5 Complete Pipe Replacement Projects
- 5 Pipe Lining Projects
- 2 Bursting Projects

- 1 Pipe Patching Project
- 2 Manhole Rehabilitation Projects
- 1 In-Pipe Repair Project

Pipe replacement is the most invasive type of repair work, where a new trench must be dug and a plan to maintain or bypass sewer service during construction implemented. Lining, bursting, and patching projects can often be done in several hours after preparation work. They are non-invasive and result in little ground disturbance, short interruptions to sewage flows, and are generally less costly. Consequently non-invasive projects were preferred when judged feasible.

Approximately 6000 feet of pipe and nearly 30 manholes have been recommended for repair or replacement. As such, not all the suspected deficiencies were fully investigated, making it likely that numerous undiscovered deficiencies remain in the system.

#### 7.1.1.2. Summary of I/I Capital Improvement Plan

A total combination of all the projects recommended in this study resulted in a cost in today's dollar of **$1,436,675.** Due to the high cost, it is not feasible for any public utility operator to complete all of their needed improvements immediately following an analysis. Therefore to better organize rehabilitation efforts by the City, the various projects were prioritized and ranked to allow the City to manage their resources and get the greatest benefit for each dollar invested in I/I rehabilitation.

The I/I Capital Improvement Plan (CIP) has been broken into four priority levels, with lower numbers reflecting the most urgent repairs.

- **Priority 1**, projects which need immediate repairs with large deficiencies and extreme I/I.
  - **Total Repairs $380,935**
- **Priority 2**, projects which need repair over the next few years. Deficiencies are not as serious as Priority 1. As such, projects may be delayed.
  - **Total Repairs $565,400**
- **Priority 3**, projects with less systemic deficiencies and more isolated I/I points. Repair is suggested before the next 5-6 years.
  - **Total Repairs $350,260**
- **Priority 4**, projects mainly needing point repairs or with minor deficiencies that were not observed contributing substantial I/I to the collections system.
  - **Total Repairs $140,080**

It is anticipated that the City will pursue funding assistance in completing the more urgent projects and, potentially, all of the projects. At a minimum, the City should seek to address the Priority 1 & 2 repairs while actively monitoring the collection system for other serious problems.

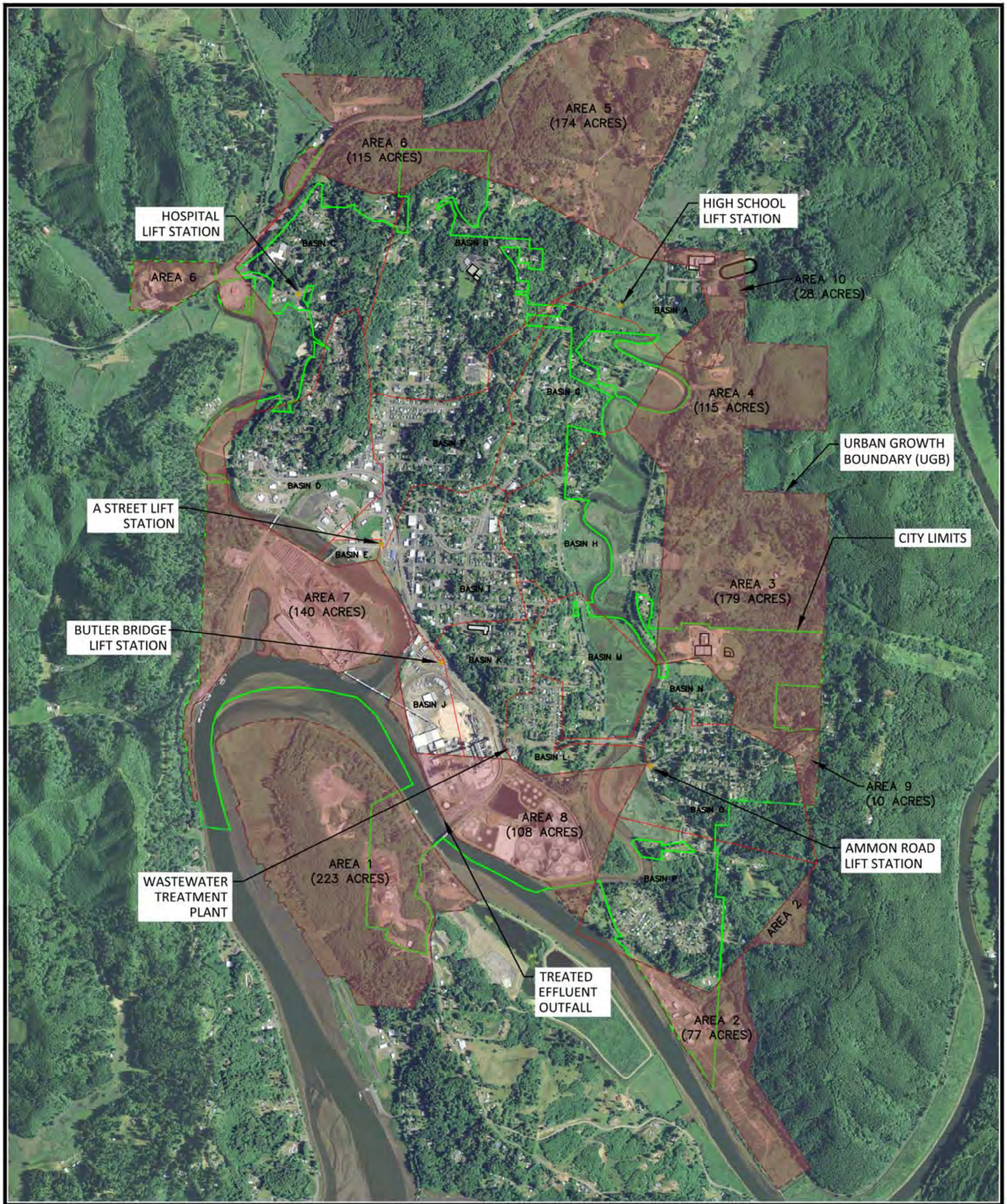
#### 7.1.1.3. General Maintenance and Continued I/I Reduction Efforts

It is believed that these high flows are the result of rain induced infiltration and the "French drain" effect of the system. During the I & I Study, it was observed that many piping sections and manholes require maintenance and cleaning. Many manholes were observed to be holding sediment and debris in the manholes. Some outlets were nearly plugged severely restricting the flow in the system. In general, the City needs to continue their efforts to reduce I/I and maintain their system. It is recommended that the City develop an annual budget category with the intention of funding I/I reduction efforts and system maintenance. As the City performs this regular maintenance on an annual basis, the need for major rehabilitation projects will be greatly reduced. DEQ recommends that pipe cleaning be a part of the ongoing general maintenance if it is not already.

### 7.1.2. **Extension of Conveyance System to Areas Currently Not Serviced with Sewer**

As part of this planning effort, it is important to discuss areas within the UGB that are currently outside the wastewater service area. As development within the service area increases, the need to extend service to these new areas will become more important. Figure 7.1.2 indicates the areas that are outside the service area of the sewer system but within the UGB. The projects discussed in this section will provide general planning for the extension of service to those areas. In some cases, the areas can be serviced through the use of gravity piping. In other cases, pumping systems will be required. An effort was made to provide preliminary cost estimates for the major “trunk” systems to service these areas. Branch piping needed to service specific projects will be developed as the need arises. It is anticipated that the funding for the expansion of the system within the UGB outlined within this report would be funded by SDC fees.

The possible new service areas within the UGB have been identified on Figure 7.1.2. A description of the basic systems that will be required to service those areas is provided below:




| | | | | |
|---|---|---|---|---|
| Civil West Engineering Services, Inc. | Drawn By: MLG<br>Date: Dec, 2012 | 0 1" | **Un-Serviced Area, UGB** | FIGURE |
| | Wastewater Facilities Plan | | City of Toledo<br>Lincoln County, OR | 7.1.2 |

### 7.1.3. Area 1: Airport Peninsula Area

The largest of the areas planned for future sewer service is located on the peninsula accessible by SE Butler Bridge Road (figure 7.1.2). The area lies within the current UGB and is bounded on the west, north, and east sides by the Yaquina River. Access to this area from the City is provided by SE Butler Bridge Road and Butler Bridge. Currently this area is characterized by sparse development with a significant amount of this area being used by a commercial logging operation. The terrain is low lying to the northwest and west, and low wooded hills through the central, southern, and eastern portion of the peninsula.

Because of topography and the Yaquina River, gravity sewer service cannot be extended to the treatment plant from this area. The area will have to become a new collection basin with a pump/lift station to deliver flows back to the treatment plant.

A preliminary layout of the potential collection system for the Airport Peninsula area has been completed to develop preliminary construction costs. A cost estimate for construction of a gravity sewer system to service Area 1 is provided below in Table 7.1.3a. A cost estimate for a new pump/lift station and force main is provided in Table 7.1.3b

**Table 7.1.3a - Cost Estimate for Gravity Collection System to serve Area 1**

| Gravity Sewer System Improvements | | | | | |
|---|---|---|---|---|---|
| **Item No.** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $55,000.00 | $55,000.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $17,000.00 | $17,000.00 |
| 4 | 8-inch PVC Gravity Sewer Piping | lf | 4,720 | $85.00 | $401,200.00 |
| 5 | 6-inch PVC Sewer Lateral Piping (assume 30' per residence) | lf | 360 | $53.00 | $19,080.00 |
| 6 | Sewer Lateral Cleanout or Connection w/Cleanout | ea | 12 | $265.00 | $3,180.00 |
| 7 | Standard Manhole | ea | 12 | $4,800.00 | $57,600.00 |
| | | Construction Total | | | $553,060.00 |
| | | Contingency (20%) | | | $110,612.00 |
| | | Subtotal | | | $663,672.00 |
| | | Engineering (20%) | | | $132,734.40 |
| | | Administrative costs (3%) | | | $19,910.16 |
| | | **Total Project Costs** | | | **$816,316.56** |

**Table 7.1.3b - Cost Estimate for Future Lift Station and Force Main to serve Area 1**

| New Lift Station and Force Main | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $84,000.00 | $84,000.00 |
| 2 | Construction Facilities/Temporary Systems/Demolition | ls | 1 | $50,000.00 | $50,000.00 |
| 3 | Duplex pumping equipment | ls | 1 | $26,500.00 | $26,500.00 |
| 4 | Control panel, VFD's, telemetry | ls | 1 | $28,000.00 | $28,000.00 |
| 5 | Wet w ell, piping, fittings, and vault lids | ls | 1 | $75,000.00 | $75,000.00 |
| 6 | On-site pow er generation equipment | ls | 1 | $43,000.00 | $43,000.00 |
| 7 | Site Electrical | ls | 1 | $75,000.00 | $75,000.00 |
| 8 | Control Building | sf | 100 | $265.00 | $26,500.00 |
| 9 | Site w ork, fencing, paving, flatw ork | ls | 1 | $16,000.00 | $16,000.00 |
| 10 | Valve and meter vault and tie in to force main | ls | 1 | $53,000.00 | $53,000.00 |
| 11 | 8-inch C-900 PVC Force Main | lf | 2,400 | $75.00 | $180,000.00 |
| | | Construction Total | | | $657,000.00 |
| | | Contingency (20%) | | | $131,400.00 |
| | | Subtotal | | | $788,400.00 |
| | | Engineering (20%) | | | $157,680.00 |
| | | Environmental Report | | | $20,000.00 |
| | | Land Acquisition Costs | | | $75,000.00 |
| | | Administrative costs (3%) | | | $23,652.00 |
| | | **Total Project Costs** | | | **$1,064,732.00** |

### 7.1.4. Area 2: Southern Yaquina River Area

This area is along the Yaquina River and is the southernmost area identified in the UGB outside of the current wastewater service area (figure 7.1.2). This area is characterized as mostly flood plain located along the river with a small section of wooded hills. The area not likely to see any major residential development but some commercial or industrial facilities could locate in this area. Due to the topography and distances, it is likely that a majority of this area will not be serviceable through gravity sewer service alone.

A preliminary investigation into the layout of the potential collection system for the Southern Yaquina area has been done to develop preliminary construction costs. It is anticipated that existing Basin P will receive and transport all wastewater from this area back into the City's collection system. A cost estimate for construction of the gravity sewer system which service Area 2 is provided below in Table 7.1.4a. A cost estimate for a new pump station and force main is provided in Table 7.1.4b.

**Table 7.1.4a - Cost Estimate for Gravity Sewer Extension to Area 2**

| Gravity Sewer System Improvements | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $35,000.00 | $35,000.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $11,000.00 | $11,000.00 |
| 3 | 8-inch PVC Gravity Sewer Piping | lf | 3,000 | $85.00 | $255,000.00 |
| 4 | 6-inch PVC Sewer Lateral Piping (assume 30' per residence) | lf | 150 | $53.00 | $7,950.00 |
| 5 | Sewer Lateral Cleanout or Connection w/Cleanout | ea | 5 | $265.00 | $1,325.00 |
| 6 | Standard Manhole | ea | 8 | $4,800.00 | $38,400.00 |
| | | Construction Total | | | $348,675.00 |
| | | Contingency (20%) | | | $69,735.00 |
| | | Subtotal | | | $418,410.00 |
| | | Engineering (20%) | | | $83,682.00 |
| | | Administrative costs (3%) | | | $12,552.30 |
| | | **Total Project Costs** | | | **$514,644.30** |

**Table 7.1.4b - Cost Estimate for Lift Station and Force Main to serve Area 2**

| New Lift Station and Force Main | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $84,000.00 | $84,000.00 |
| 2 | Construction Facilities/Temporary Systems/Demolition | ls | 1 | $50,000.00 | $50,000.00 |
| 3 | Duplex pumping equipment | ls | 1 | $26,500.00 | $26,500.00 |
| 4 | Control panel, VFD's, telemetry | ls | 1 | $28,000.00 | $28,000.00 |
| 5 | Wet well, piping, fittings, and vault lids | ls | 1 | $75,000.00 | $75,000.00 |
| 6 | On-site power generation equipment | ls | 1 | $43,000.00 | $43,000.00 |
| 7 | Site Electrical | ls | 1 | $75,000.00 | $75,000.00 |
| 8 | Control Building | sf | 100 | $265.00 | $26,500.00 |
| 9 | Site work, fencing, paving, flatwork | ls | 1 | $16,000.00 | $16,000.00 |
| 10 | Valve and meter vault and tie in to force main | ls | 1 | $53,000.00 | $53,000.00 |
| 11 | 8-inch C-900 PVC Force Main | lf | 1,200 | $75.00 | $90,000.00 |
| | | Construction Total | | | $567,000.00 |
| | | Contingency (20%) | | | $113,400.00 |
| | | Subtotal | | | $680,400.00 |
| | | Engineering (20%) | | | $136,080.00 |
| | | Environmental Report | | | $20,000.00 |
| | | Land Acquisition Costs | | | $75,000.00 |
| | | Administrative costs (3%) | | | $20,412.00 |
| | | **Total Project Costs** | | | **$931,892.00** |

### 7.1.5. Area 3: Southern Sturdevant Road Area

This area is along the eastern UGB just south of the power substation and north of the Toledo Middle School but outside of the wastewater service area (figure 7.1.2). This area is characterized as wooded hilly with multiple residences on small acreages. The area is attractive and will likely see development pressures as opportunities within the current service area diminish. Due to the topography and distances, it is likely that this area will be serviceable through gravity sewer service to Basin N.

A preliminary investigation into the layout of the potential collection system for the Southern Sturdevant Road area has been done to develop preliminary construction costs. A cost estimate for construction of a gravity sewer system to service Area 3 is provided below in Table 7.1.5.

**Table 7.1.5 - Cost Estimate for Gravity Sewer Extension to Area 3**

| Gravity Sewer System Improvements | | | | | |
|---|---|---|---|---|---|
| **Item No.** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $85,000.00 | $85,000.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $28,000.00 | $28,000.00 |
| 3 | 10-inch PVC Gravity Sewer Piping | lf | 1,800 | $95.00 | $171,000.00 |
| 4 | 8-inch PVC Gravity Sewer Piping | lf | 5,323 | $85.00 | $452,455.00 |
| 5 | 6-inch PVC Sewer Lateral Piping (assume 30' per residence) | lf | 720 | $53.00 | $38,160.00 |
| 6 | Sewer Lateral Cleanout or Connection w/Cleanout | ea | 24 | $265.00 | $6,360.00 |
| 7 | Standard Manhole | ea | 18 | $4,800.00 | $86,400.00 |
| | | Construction Total | | | $867,375.00 |
| | | Contingency (20%) | | | $173,475.00 |
| | | Subtotal | | | $1,040,850.00 |
| | | Engineering (20%) | | | $208,170.00 |
| | | Administrative costs (3%) | | | $31,225.50 |
| | | **Total Project Costs** | | | **$1,280,245.50** |

### 7.1.6. Area 4: Central Sturdevant Road Area

This area is along the eastern UGB just south of the Toledo High School and north of the power substation but outside of the wastewater service area (figure 7.1.2). This area is characterized as wooded hilly with multiple residences on small acreages. The area is attractive and will likely see development pressures as opportunities within the current service area diminish. Due to the topography and distances, it is likely that this area will not be serviceable through gravity sewer service to Basin G, H or N.

A preliminary investigation into the layout of the potential collection system for the Central Sturdevant Road area has been done to develop preliminary construction costs. A cost estimate for construction of a gravity sewer system to service Area 4 is provided below in Table 7.1.6a. A cost estimate for a new pump station and force main is provided in Table 7.1.6b.

**Table 7.1.6a - Cost Estimate for Gravity Sewer Extension to Area 4**

| Gravity Sewer System Improvements | | | | | |
|---|---|---|---|---|---|
| **Item No.** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $45,000.00 | $45,000.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $13,000.00 | $13,000.00 |
| 3 | 8-inch PVC Gravity Sewer Piping | lf | 3,700 | $85.00 | $314,500.00 |
| 4 | 6-inch PVC Sewer Lateral Piping (assume 30' per residence) | lf | 360 | $53.00 | $19,080.00 |
| 5 | Sewer Lateral Cleanout or Connection w/Cleanout | ea | 12 | $265.00 | $3,180.00 |
| 6 | Standard Manhole | ea | 10 | $4,800.00 | $48,000.00 |
| | | Construction Total | | | $442,760.00 |
| | | Contingency (20%) | | | $88,552.00 |
| | | Subtotal | | | $531,312.00 |
| | | Engineering (20%) | | | $106,262.40 |
| | | Administrative costs (3%) | | | $15,939.36 |
| | | **Total Project Costs** | | | **$653,513.76** |

**Table 7.1.6b - Cost Estimate for Lift Station and Force Main to serve Area 4**

| New Lift Station and Force Main | | | | | |
|---|---|---|---|---|---|
| **Item No.** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $84,000.00 | $84,000.00 |
| 2 | Construction Facilities/Temporary Systems/Demolition | ls | 1 | $50,000.00 | $50,000.00 |
| 3 | Duplex pumping equipment | ls | 1 | $26,500.00 | $26,500.00 |
| 4 | Control panel, VFD's, telemetry | ls | 1 | $28,000.00 | $28,000.00 |
| 5 | Wet w ell, piping, fittings, and vault lids | ls | 1 | $75,000.00 | $75,000.00 |
| 6 | On-site pow er generation equipment | ls | 1 | $43,000.00 | $43,000.00 |
| 7 | Site Electrical | ls | 1 | $75,000.00 | $75,000.00 |
| 8 | Control Building | sf | 100 | $265.00 | $26,500.00 |
| 9 | Site w ork, fencing, paving, flatw ork | ls | 1 | $16,000.00 | $16,000.00 |
| 10 | Valve and meter vault and tie in to force main | ls | 1 | $53,000.00 | $53,000.00 |
| 11 | 8-inch C-900 PVC Force Main | lf | 900 | $75.00 | $67,500.00 |
| | | Construction Total | | | $544,500.00 |
| | | Contingency (20%) | | | $108,900.00 |
| | | Subtotal | | | $653,400.00 |
| | | Engineering (20%) | | | $130,680.00 |
| | | Environmental Report | | | $20,000.00 |
| | | Land Acquisition Costs | | | $75,000.00 |
| | | Administrative costs (3%) | | | $19,602.00 |
| | | **Total Project Costs** | | | **$898,682.00** |

### 7.1.7. Area 5: Northern Olalla Slough Area

This area is along the Northern UGB just southeast of Hwy 20 and northwest of the Olalla Slough, but outside of the wastewater service area (figure 7.1.2). This area is characterized as wooded hilly with multiple residential on small acreages. The area is attractive and will likely see development pressures as opportunities within the current service area diminish. Due to the topography and distances, it is likely that this area will be serviceable through gravity sewer service to Basin A. Basin A currently drains to the High School Lift Station which is unlikely to have the capacity to service this area without an upgrade to the lift station or the construction of a new lift station.

A preliminary investigation into the layout of the potential collection system for the Northern Olalla Slough area has been done to develop preliminary construction costs. A cost estimate for construction of a gravity sewer system to service Area 5 is provided below in Table 7.1.7a. A cost estimate for a new pump station and force main to replace the existing High School Lift Station is provided in Table 7.1.7b.

**Table 7.1.7a - Cost Estimate for Gravity Sewer Extension to Area 5**

| Gravity Sewer System Improvements | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $120,000.00 | $120,000.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $40,000.00 | $40,000.00 |
| 3 | 10-inch PVC Gravity Sewer Piping | lf | 2,000 | $95.00 | $190,000.00 |
| 4 | 8-inch PVC Gravity Sewer Piping | lf | 7,950 | $85.00 | $675,750.00 |
| 5 | 6-inch PVC Sewer Lateral Piping (assume 30' per residence) | lf | 1,050 | $53.00 | $55,650.00 |
| 6 | Sewer Lateral Cleanout or Connection w/Cleanout | ea | 35 | $265.00 | $9,275.00 |
| 7 | Standard Manhole | ea | 25 | $4,800.00 | $120,000.00 |
| | | Construction Total | | | $1,210,675.00 |
| | | Contingency (20%) | | | $242,135.00 |
| | | Subtotal | | | $1,452,810.00 |
| | | Engineering (20%) | | | $290,562.00 |
| | | Administrative costs (3%) | | | $43,584.30 |
| | | **Total Project Costs** | | | **$1,786,956.30** |

**Table 7.1.7b - Cost Estimate for Replacing High School Lift Station to serve Area 5**

| New Lift Station and Force Main | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $84,000.00 | $84,000.00 |
| 2 | Construction Facilities/Temporary Systems/Demolition | ls | 1 | $50,000.00 | $50,000.00 |
| 3 | Duplex pumping equipment | ls | 1 | $26,500.00 | $26,500.00 |
| 4 | Control panel, VFD's, telemetry | ls | 1 | $28,000.00 | $28,000.00 |
| 5 | Wet well, piping, fittings, and vault lids | ls | 1 | $75,000.00 | $75,000.00 |
| 6 | On-site power generation equipment | ls | 1 | $43,000.00 | $43,000.00 |
| 7 | Site Electrical | ls | 1 | $75,000.00 | $75,000.00 |
| 8 | Control Building | sf | 100 | $265.00 | $26,500.00 |
| 9 | Site work, fencing, paving, flatwork | ls | 1 | $16,000.00 | $16,000.00 |
| 10 | Valve and meter vault and tie in to force main | ls | 1 | $53,000.00 | $53,000.00 |
| 11 | 8-inch C-900 PVC Force Main | lf | 2,100 | $75.00 | $157,500.00 |
| 12 | 10-inch PVC Gravity Sewer Piping | lf | 1,100 | $95.00 | $104,500.00 |
| 13 | Standard Manhole | ea | 3 | $4,800.00 | $14,400.00 |
| | | Construction Total | | | $753,400.00 |
| | | Contingency (20%) | | | $150,680.00 |
| | | Subtotal | | | $904,080.00 |
| | | Engineering (20%) | | | $180,816.00 |
| | | Environmental Report | | | $20,000.00 |
| | | Land Acquisition Costs | | | $75,000.00 |
| | | Administrative costs (3%) | | | $27,122.40 |
| | | **Total Project Costs** | | | **$1,207,018.40** |

### 7.1.8. **Area 6: Hwy 20 Area**

This area is along the northwestern UGB along Hwy 20 from the Depot Slough on the southwest to Arcadia Drive on the northeast (figure 7.1.2). This area is characterized as wooded hilly with low to medium density residential homes and some small commercial facilities already spread throughout the area. The area is attractive and will likely see development pressures as opportunities within the current service area diminish. Due to the topography and distances, it is likely that this area will be serviceable through gravity sewer service to Basin C.

A preliminary investigation into the layout of the potential collection system for the Hwy 20 area has been done to develop preliminary construction costs. A cost estimate for construction of a gravity sewer system to service Area 6 is provided below in Table 7.1.8.

**Table 7.1.8. Cost Estimate for Gravity Sewer Extension to Area 6**

| Gravity Sewer System Improvements | | | | | |
|---|---|---|---|---|---|
| **Item No.** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $69,000.00 | $69,000.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $23,000.00 | $23,000.00 |
| 3 | 8-inch PVC Gravity Sewer Piping | lf | 6,100 | $85.00 | $518,500.00 |
| 4 | 6-inch PVC Sewer Lateral Piping (assume 30' per residence) | lf | 300 | $53.00 | $15,900.00 |
| 5 | Sewer Lateral Cleanout or Connection w/Cleanout | ea | 10 | $265.00 | $2,650.00 |
| 6 | Standard Manhole | ea | 16 | $4,800.00 | $76,800.00 |
| | | Construction Total | | | $705,850.00 |
| | | Contingency (20%) | | | $141,170.00 |
| | | Subtotal | | | $847,020.00 |
| | | Engineering (20%) | | | $169,404.00 |
| | | Administrative costs (3%) | | | $25,410.60 |
| | | **Total Project Costs** | | | **$1,041,834.60** |

7.1.9.**Area 7: Sawmill Area**

This area is on the west side of the UGB where the Depot Slough intersects with the Yaquina River (figure 7.1.2). This area is characterized flat land that is currently zoned commercial. A majority of this area is currently occupied by a saw mill. This area is expected to be provided service by gravity sewer service to Basin E.

A preliminary investigation into the layout of the potential collection system for the Sawmill area has been done to develop preliminary construction costs. A cost estimate for construction of a gravity sewer system to service Area 7 is provided below in Table 7.1.9.

**Table 7.1.9. Cost Estimate for Gravity Sewer Extension to Area 7**

| Gravity Sewer System Improvements | | | | | |
|---|---|---|---|---|---|
| **Item No.** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $21,000.00 | $21,000.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $7,800.00 | $7,800.00 |
| 3 | 8-inch PVC Gravity Sewer Piping | lf | 1,800 | $85.00 | $153,000.00 |
| 4 | 6-inch PVC Sewer Lateral Piping (assume 30' per residence) | lf | 60 | $53.00 | $3,180.00 |
| 5 | Sewer Lateral Cleanout or Connection w/Cleanout | ea | 2 | $265.00 | $530.00 |
| 6 | Standard Manhole | ea | 5 | $4,800.00 | $24,000.00 |
| | | Construction Total | | | $209,510.00 |
| | | Contingency (20%) | | | $41,902.00 |
| | | Subtotal | | | $251,412.00 |
| | | Engineering (20%) | | | $50,282.40 |
| | | Administrative costs (3%) | | | $7,542.36 |
| | | **Total Project Costs** | | | **$309,236.76** |

#### 7.1.10. **Area 8, 9, and 10: Currently Developed; Not Requiring Major Improvements**

These areas include a wide range of terrain located in several locations within the UGB. All of these areas are outside the currently defined sewer basins; some have existing improvements such as the paper mill along the Yaquina River. All of these areas have one thing in common and that is the need to construct major trunk sewer lines or lift stations to provide service to the areas. These areas can either be serviced through branch sewer line extensions from the existing sewer collection system, or, in the instance of the paper mill area, the likelihood of the property needing sewer service is not high. Therefore no specific investigations or cost estimates were prepared for these areas.

## *7.2. Lift Station Alternatives*

Many of the lift stations are currently distressed due to differential settling between the wetwell/drywell and the generator housing. Designs of any upgrades or replacements will need to be designed appropriately to alleviate this common problem.

### 7.2.1. **A Street Lift Station**

See section 4 for discussion on the current condition of the A Street Lift Station. A significant improvement project will be required at the A Street Lift Station in order to address the existing problems.

Investigations into the current and projected flows for this lift station have resulted in the following peak instantaneous flows that this facility must be capable of handling:

Current Total Peak Instantaneous Flow ..................................................... 2.77 mgpd (1,923 gpm)
Projected Peak Instantaneous Flow ............................................................ 3.22 mgpd (2,236 gpm)

As stated in Section 6, pump stations must be designed to handle the peak instantaneous flows.

Therefore, based on this analysis, the A Street Lift Station needs be able to handle a projected firm pumping capacity of 2,236 gpm. This can be accomplished with approximately two 2,240 gpm pumps (duplex) or three 1,120 gpm pumps (triplex).

The recommended wet well volume for this facility is defined by two basic criteria. The first, the facility must be designed to prevent excessive number of pump starts per hour. Pump manufacturers typically recommend a maximum of 15 starts per hour and designing for approximately 10 starts per hour. For constant speed pumps, the minimum wet well volume between low water level (LWL) and pump on level can be calculated using the following formula:

$$V_{minimum} = (T_{minutes} \times Q_{max}) / 4$$

$V_{minimum}$ = Minimum volume in cubic feet
$T_{minutes}$ = Target time between pump starts in minutes (10 starts per hour or 6 minutes)
$Q_{max}$ = Pump design capacity, use 2,240 gpm (299.4 $ft^3$/minute)

Therefore: $V_{minimum} = (6 \text{ minutes} \times 299.4\ ft^3/\text{minute}) / 4 = 449.1 ft^3$ (3,360 Gallons)

The second criteria used to define wet well volume identify the maximum storage volume allowed while avoiding septic conditions within the wet well. In general, average detention time should be no more than 35 minutes during average flow conditions during the dry season. The average maximum wet well volume required to avoid septic conditions can be calculated as follows:

$V_{wetwell} = Q_{summer}$ x 35 minutes

$V_{wetwell}$ = Maximum wetwell volume to avoid septic conditions
$Q_{summer}$ = Dry season average flow (Approximate) = 169 gpm

Therefore: $V_{wetwell}$ = 169 gpm x 35 minutes = 5,915 gallons (790.7 $ft^3$)

Based on these calculations a properly sized wet well for the A Street lift station should have a minimum wet well storage volume of 3,360 gallons and a maximum storage volume of 5,915 gallons. These limits will prevent excessive pump starts which can increase the wear on the pump stations pumps as well as limit the detention time preventing the development of septic conditions within the wet well.

To address the deficiencies at this lift station improvement alternatives were developed and are discussed below for the A Street Lift Station. A "do nothing" alternative will likely result in untreated overflows due to significant flow and storage deficiency and the poor condition of the above ground structure.

**Table 7.2.1 - A Street Lift Station Data**

| A Street Lift Station | |
|---|---|
| Location | 1st Street and 'A' Street |
| Type of Station | Wet well / dry well, duplex, constructed in 1954, pumps replaced within past 10 years |
| Pump Type | Non-clog, centrifugal pump |
| Motor Data | 20 Hp |
| Firm Capacity | Approximately 820 gpm |
| Overflow Point | Overflow is at manhole F-2, the elevation is unknown. |
| Overflow Discharge | Discharges to Depot Slough. |
| Auxiliary Power | On-Site automatic transfer switch 80 KW diesel generator with 50 gallon fuel capacity. |
| Current Flows | Current PIF is approximately 1,674 gpm. |
| Projected Flow | The 20 year projected PIF is 1,945 gpm. |
| Projected Capacity | This pump station is undersized and needs to be replaced during the planning period. |

7.2.1.1. A Street Lift Station – Dry well Upgrade

Because the existing station is a dry well/wet well type station, capacity to the station could be increased through the installation of new pumps in the dry well. It is becoming increasingly common to install submersible solids handling pumps in a dry well configuration. This provides the advantages of submersible solids handling capabilities and reliabilities with the ease of installation of a dry well pump.

The disadvantages of continuing to operate the station as a dry well pump station are numerous. Firstly, the deep dry and wet wells are considered confined spaces which necessitate special safety measures for anyone entering the pits. Harnesses, hoists, ventilation, gas detection, multiple personnel, and other considerations must be met before anyone can enter the pits to perform maintenance or observe the operation of the pumps.

Also, because the station is over 50-years of age, much of the internal components are worn and would require replacement. This could include pipe and fittings, valves, hooks, tie-offs, access ladders, and the above ground buildings housing the controls and the backup generator.

A significant disadvantage to continuing to operate the station as a dry well/wet well station is the limitations in the wet well capacity. The existing wet well can hold 284 gal per foot of depth; at the 3 foot range from the existing pump on-pump off switches the well has a storage capacity of 853 gallons

(114.0 ft$^3$). As defined above this facility should provide a minimum of 3,360 gallons of storage which is significantly more than the current wet well capacity. This deficiency will accelerate the wear on the pumps increasing the maintenance and repairs required over the useful life of the facility.

This alternative would require the installation of two new pumps in the existing lift station, each capable of 2,240 gpm. While lower flows can be addressed by using VFD's, or a smaller pump appropriately sized to handle smaller flows, the small wet well will result in less operating flexibility and more starts and stops on the pumps increasing the likelihood of maintenance for the facility.

A preliminary cost estimate is provided below for the dry well upgrade alternative:

**Table 7.2.1.1 - A Street Lift Station Upgrades – Dry well Upgrade Cost Estimate**

| A Street Lift Station Improvements – Dry Pit Upgrade | | | | | |
|---|---|---|---|---|---|
| Item | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | LS | 1 | $41,000 | $41,000 |
| 2 | Construction Facilities/Temporary Systems/Demolition | LS | 1 | $25,000 | $25,000 |
| 3 | Bypass pumping | LS | 1 | $11,500 | $11,500 |
| 4 | New duplex pumps and equipment | EA | 2 | $67,000 | $134,000 |
| 5 | Control panel, VFD's, telemetry (explosion proof) | LS | 1 | $40,000 | $40,000 |
| 6 | Piping and fitting upgrades in pits | LS | 1 | $28,000 | $28,000 |
| 7 | Concrete coating and repair in pits | LS | 1 | $17,000 | $17,000 |
| 8 | Electrical improvements-intrinsically safe | LS | 1 | $95,000 | $95,000 |
| 9 | Control and Generator Building improvements | LS | 1 | $22,500 | $22,500 |
| 10 | Flow meter vault and force main tie-in | LS | 1 | $34,000 | $34,000 |
| | | Construction Total | | | $448,000 |
| | | Contingency (20%) | | | $89,600 |
| | | Subtotal | | | $537,600 |
| | | Engineering (20%) | | | $107,520 |
| | | Environmental Report | | | $10,000 |
| | | Administrative costs (3%) | | | $16,128 |
| | | **Total Project Costs** | | | **$671,248** |

7.2.1.2. A Street Lift Station – New Wet Well

The City has indicated a desire to eliminate the confined space and explosion issues related to the current wet well/dry well station. The simplest way to accomplish this is to construct a new pump station wet well adjacent to the existing pump station and install new submersible pumps in the wet well. Construction of a new wet well adjacent to and between the existing station and City owned building west of the current lift station could be possible, and may not require acquisition of additional property.

The new wet well could be set up as a tri-plex wet well to increase greater operational flexibility. This would allow the City to install two pumps now with variable speed drives (each capable of the firm pumping capacity of 2,240 gpm) and adding a third in the future should the need arise. A preferred option would be to install three smaller pumps (1,120 gpm each) to meet the capacity and redundancy requirements. A triplex configuration would be better able to accommodate potential increases in flow beyond the 20-year planning period. Another option would be to install a smaller duty pump to handle the lower flows. For the purpose of this evaluation, two full size pumps are initially required.

As identified previously, this facility should provide a minimum storage capacity of 3,360 gallons and a maximum storage capacity of 5,915 gallons. By selection to use the 5,915 gallons storage capacity this facility has the ability to adequately address current as well as future flows while helping to minimize the chance for an overflow event. Assuming that the new wet well will be 8 foot diameter the required volume of storage between the pump on and pump off switch would be 5,915 gallons (790.7 $ft^3$). This equates to approximately 16 feet between the switches. To prevent a backup into the collection piping the high water alarm should be set approximately 1 foot above the pump on switch and 1 foot below the invert into the wet well. It is also assumed that a minimum of 2 feet of depth will be maintained below the facility's storage volume to ensure the pump intakes are adequately covered. The existing lift station has a pipe inlet invert of approximately 3 feet below ground surface; using this as the pipe inlet invert in the new facility the total depth of the wet well will be approximately 23 feet from ground surface. This configuration will provide adequate capacity within the wet well so that during peak flow periods the system would no longer surcharge back into the collection network and overflow into the nearby slough.

Reusing the existing wetwell and drywell by removing the center wall is not recommended based on structural concerns of the concrete and the possible exacerbation of any existing damage doing the significant demolition of the center wall.

The existing above ground structure is in poor condition therefore a new building will be required to house the new electrical and control equipment and backup generator. A building approximately 10 feet x 14 feet should be adequate.

A preliminary cost estimate for this alternative is provided below:

**Table 7.2.1.2 - A Street Lift Station Upgrades – New Lift Station Cost Estimate**

| A Street Lift Station – New Lift Station | | | | | |
|---|---|---|---|---|---|
| **Item** | **Description** | **Unit** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Mobilization, Insurance, Overhead, Bonds (10%) | LS | 1 | $110,389 | $110,389 |
| 2 | Construction Facilities/Temporary Systems/Bypass Provisions | LS | 1 | $35,000 | $35,000 |
| 3 | Wetw ell w ith Polyurea Coating, Excavation, Installation | LS | 1 | $210,000 | $210,000 |
| 4 | 100 HP Pump, VFD, Accessories and Installation | EA | 2 | $85,000 | $170,000 |
| 5 | Electrical, Wiring, Panels, Level Controls, SCADA | LS | 1 | $90,000 | $90,000 |
| 6 | Relocate Generator, Fuel Supply, ATS, Ventilation & Ducting | LS | 1 | $8,250 | $8,250 |
| 7 | Control & Generator Building w /Dividing Wall & Rollup Door | LS | 1 | $85,000 | $85,000 |
| 8 | Site Piping, Valves, Fittings and Vault | LS | 1 | $60,000 | $60,000 |
| 9 | Flow meter and Vault | LS | 1 | $18,000 | $18,000 |
| 10 | 8-Inch Influent Pipe | LF | 20 | $125 | $2,500 |
| 11 | Site Work | LS | 1 | $20,000 | $20,000 |
| 12 | 12" Force main | LF | 20 | $233 | $4,660 |
| 13 | New Manhole | EA | 1 | $4,500 | $4,500 |
| 14 | Demolition and Abandonment of Lift Station | LS | 1 | $24,750 | $24,750 |
| 15 | Misc. Restoration and Clean Up | LS | 1 | $15,000 | $15,000 |
| | | Construction Total | | | $858,049 |
| | | Contingency (20%) | | | $171,610 |
| | | Subtotal | | | $1,029,659 |
| | | Engineering (20%) | | | $205,932 |
| | | Environmental Report | | | $20,000 |
| | | Environmental Engineering* | | | $40,000 |
| | | Administrative Costs (3%) | | | $30,890 |
| | | **Total Project Cost** | | | $1,326,480 |

*If needed

### 7.2.1.3. A Street Lift Station - Force Main

The 8-inch force main for the A Street lift station is over 50 years old and constructed out of asbestos cement (AC) pipe. The force main is routed down 1st Street to Manhole No. I-2 where it discharges into the gravity collection system serviced by the Butler Bridge Lift Station. At 2,240 gpm, the velocity in an 8-inch force main is nearly 14.3 ft/s well above the desirable limits. Therefore, due to the combination of the age of the force main and the high velocities from the upgraded pump station it is recommended that the force main be replaced when the lift station is reconstructed. A 10-inch force main would have maximum velocities at 2,240 gpm slightly above 9 ft/s, which is marginally higher than the DEQ recommended velocity for a force main. Therefore, a new 12-inch force main is recommended which will have a velocity of just above 6.3 ft/s at a flow rate of 2,240 gpm.

The receiving Manhole No. I-2 (structural integrity unknown) appears to have an 18-inch gravity pipe that extends south along Butler Bridge Road to the Butler Bridge lift station. With the existing 18 trunk line available to accept the flows from the A Street lift it is not anticipated that the gravity piping would be overwhelmed. Therefore, the new force main could stay in its existing alignment along 1st Street. Prior to design, a detailed inspection of the discharge manhole (I-2) should be performed. Based on its structural integrity and the amount of corrosion identified, it may need to be coated or replaced.

The current force main alignment coupled with its short length make traditional open trench construction of the new 12-inch force main parallel to the existing force main the most cost effective and appropriate means of construction. A preliminary cost estimate for this alternative is provided below:

**Table 7.2.1.3 - A Street Force Main – Open Trench Construction Cost Estimate**

| A Street Lift Station - New 12-Inch Force Main | | | | | |
|---|---|---|---|---|---|
| **Item** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Mobilization, Insurance, Overhead, Bonds (10%) | LS | 1 | $29,000 | $29,000 |
| 2 | Construction Facilities, Temporary Systems and Bypass Provisions | LS | 1 | $23,000 | $23,000 |
| 3 | New 12-Inch HDPE Force Main | LF | 250 | $233 | $58,250 |
| 4 | Tie ins, Manhole Connections, Fittings, etc. | ea | 1 | $6,400 | $6,400 |
| | | Construction Total | | | $116,650 |
| | | Contingency (20%) | | | $23,330 |
| | | Subtotal | | | $139,980 |
| | | Engineering (20%) | | | $27,996 |
| | | Administrative Costs (3%) | | | $4,199 |
| | | **Total Project Cost** | | | **$172,175** |

### 7.2.1.4. A Street Lift Station - Summation and Recommendations

A number of options for the improvements to the A Street Lift Station and its force main have been discussed above. These included upgrading the station as a dry well station which is anticipated to be less expensive than the construction of a new wet well style facility. The updates to the dry well station will require significant improvements to the pit areas which are considered hazardous spaces. It is recognized that the City wishes to eliminate confined space entry requirements for the A Street Lift Station but at this time the most cost effective way to improve the facility and meet the future needs of this area is with a modification to the existing lift station.

Therefore, it is recommended that the City undertake a project to update the current wet/dry wells with new pumps, new controls, and a new building at the existing lift station site. This will provide the City with an updated lift station capable of addressing current flows as well as future flows as the community continues to expand while minimizing the costs associated with this facility upgrade.

With the updated lift station, a force main upgrade will also be required. While there are several construction options for the installation of the force main it is recommended that the City utilizes open trench construction to minimize construction costs. This will also allow the existing force main to operate during construction and will help to minimize the overall cost associated with updating the A Street Lift Station and its force main.

### 7.2.2. **Ammon Road Lift Station**

See section 4 for discussion on the current condition of the Ammon Road Lift Station. A significant improvement project will be required at the Ammon Road Lift Station in order to address the existing problems.

Investigations into the current and projected flows for this lift station have resulted in the following peak instantaneous flows that this facility must be capable of handling:

Current Total Peak Instantaneous Flow .................................................... 1.75 mgpd (1,215 gpm)
Projected Peak Instantaneous Flow ........................................................ 1.98 mgpd (1,375 gpm)

As stated in Section 6, lift stations must be designed to handle the peak instantaneous flows.

Therefore, based on this analysis, the Ammon Road Lift Station needs be able to handle a projected firm pumping capacity of 1,375 gpm. This can be accomplished with approximately two 1,400 gpm pumps (duplex) or three 700 gpm pumps (triplex).

The recommended wet well volume for this facility is defined by two basic criteria. The first, the facility must be designed to prevent excessive number of pump starts per hour. Pump manufacturers typically recommend a maximum of 15 starts per hour and designing for approximately 10 starts per hour. For constant speed pumps, the minimum wet well volume between low water level (LWL) and pump on level can be calculated using the following formula:

$$V_{minimum} = (T_{minutes} \times Q_{max}) / 4$$

$V_{minimum}$ = Minimum volume in cubic feet
$T_{minutes}$ = Target time between pump starts in minutes (10 starts per hour or 6 minutes)
$Q_{max}$ = Pump design capacity, use 1,400 gpm (187.2 $ft^3$/minute)

Therefore: $V_{minimum} = (6 \text{ minutes} \times 187.2\ ft^3/\text{minute}) / 4 = 280.8\ ft^3$ (2,100 gallons)

The second criteria used to define wet well volume identify the maximum storage volume allowed while avoiding septic conditions within the wet well. In general, average detention time should be no more than 35 minutes during average flow conditions during the dry season. The average maximum wet well volume required to avoid septic conditions can be calculated as follows:

$$V_{wetwell} = Q_{summer} \times 35 \text{ minutes}$$

$V_{wetwell}$ = Maximum wetwell volume to avoid septic conditions
$Q_{summer}$ = Dry season average flow (Approximate) = 107 gpm

Therefore: $V_{wetwell}$ = 107 gpm x 35 minutes = 3,745 gallons (500.6 $ft^3$)

Based on these calculations a properly sized wet well for the A Street lift station should have a minimum wet well storage volume of 2,100 gallons and a maximum storage volume of 3,745 gallons. These limits will prevent excessive pump starts which can increase the wear on the pump stations pumps as well as limit the detention time preventing the development of septic conditions within the wet well.

To address the deficiencies at this lift station improvement alternatives were developed and are discussed below for the Ammon Road Lift Station. A "do nothing" alternative is not an option for this lift station due to the significant flow and storage deficiency and the poor condition of the above ground structure.

**Table 7.2.2. Ammon Road Lift Station Data**

| **Ammon Road Lift Station** | |
|---|---|
| Location | Sturdevant Road, between Ammon Road and Alder Lane |
| Type of Station | Wet well / dry well, duplex flooded suction |
| Pump Type | Non-clog, centrifugal pump |
| Motor Data | 50 Hp |
| Firm Capacity | Approximately 820 gpm |
| Overflow Point | Overflow is at manhole N-5, the elevation in unknown. |
| Overflow Discharge | Discharges to Olalla Slough. |
| Auxiliary Power | On-Site automatic transfer switch 80 KW diesel generator with 50 gallon fuel capacity. |
| Current Flows | Current PIF are approximately 1,215 gpm. |
| Projected Flow | The 20 year projected PIF is 1,375 gpm. |
| Projected Capacity | This pump station is undersized and needs to be replaced during the planning period. |

7.2.2.1. Ammon Road Lift Station – Dry well Upgrade

Because the existing station is a dry well/wet well type station, capacity to the station could be increased through the installation of new pumps in the dry well. It is becoming increasingly common to install submersible solids handling pumps in a dry well configuration. This provides the advantages of submersible solids handling capabilities and reliabilities with the ease of installation of a dry well pump.

The disadvantages of continuing to operate the station as a dry well lift station are numerous. Firstly, the deep dry and wet wells are considered confined spaces which necessitate special safety measures for anyone entering the pits. Harnesses, hoists, ventilation, gas detection, multiple personnel, and other considerations must be met before anyone can enter the pits to perform maintenance or observe the operation of the pumps.

Also, because the station is over 50-years of age, much of the internal components are worn and would require replacement. This could include pipe and fittings, valves, hooks, tie-offs, access ladders, and the above ground buildings housing the controls and the backup generator.

A significant disadvantage to continuing to operate the station as a dry well/wet well station is the limitations in the wet well capacity. The existing wet well can hold 284 gal per foot of depth; at the 3 foot range from the existing pump on-pump off switches the well has a storage capacity of 853 gallons (114.0 $ft^3$). As defined above this facility should provide a minimum of 2,100 gallons of storage which is significantly more than the current wet well capacity. This deficiency will accelerate the wear on the pumps increasing the maintenance and repairs required over the useful life of the facility.

If this alternative is selected, the City, at a minimum must install two new pumps in the existing lift station, each capable of 1,400 gpm, as well as address the leaking divider wall in the facility. While lower flows can be addressed by using VFD's, the small wet well will result in less operating flexibility and more starts and stops on the pumps increasing the likelihood of maintenance for the facility.

A preliminary cost estimate is provided below for the dry well upgrade alternative:

**Table 7.2.2.1. Ammon Road Lift Station Upgrades – Dry well Upgrade Cost Estimate**

| Ammon Road Lift Station Improvements – Dry Pit Upgrade | | | | | |
|---|---|---|---|---|---|
| Item | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | LS | 1 | $41,000 | $41,000 |
| 2 | Construction Facilities/Temporary Systems/Demolition | LS | 1 | $25,000 | $25,000 |
| 3 | Bypass pumping | LS | 1 | $11,500 | $11,500 |
| 4 | New duplex pumping equipment | EA | 2 | $67,000 | $134,000 |
| 5 | Control panel, VFD's, telemetry (explosion proof) | LS | 1 | $40,000 | $40,000 |
| 6 | Piping and fitting upgrades in pits | LS | 1 | $28,000 | $28,000 |
| 7 | Concrete coating and repair in pits | LS | 1 | $17,000 | $17,000 |
| 8 | Electrical improvements-intrinsically safe | LS | 1 | $95,000 | $95,000 |
| 9 | Control and Generator Building improvements | LS | 1 | $22,500 | $22,500 |
| | | Construction Total | | | $414,000 |
| | | Contingency (20%) | | | $82,800 |
| | | Subtotal | | | $496,800 |
| | | Engineering (20%) | | | $99,360 |
| | | Environmental Report | | | $10,000 |
| | | Administrative costs (3%) | | | $14,904 |
| | | **Total Project Costs** | | | **$621,064** |

7.2.2.2. Ammon Road Lift Station – New Wet Well

The City has indicated a desire to eliminate the confined space and explosion issues related to the current wet well/dry well station. The simplest way to accomplish this is to construct a new pump station wet well adjacent to the existing pump station and install new submersible pumps in the wet well. Construction of a new wet well adjacent to and between the existing station and SE Alder Lane could be possible but would most likely require acquisition of additional property.

The new wet well could be set up as a tri-plex wet well to provide greater operational flexibility. This would allow the City to install two pumps now (each capable of the firm pumping capacity of 1,400 gpm) and adding a third in the future should the need arise. A preferred option would be to install three smaller pumps (700 gpm each) to meet the capacity and redundancy requirements. A triplex configuration would be better able to accommodate potential increases in flow beyond the 20-year planning period. For the current estimates, two pumps are used. Recent advancements in pump design allows modern pumps to be run at lower levels while maintaining their ability to pass solids and not "rag up". This decision will be vetted during the pre-design and design processes.

As identified previously, this facility should provide a minimum storage capacity of 2,100 gallons and a maximum storage capacity of 3,745 gallons. By selection to use the 3,745 gallons storage capacity this facility has the ability to adequately address current as well as future flows while helping to minimize the chance for an overflow event. Assuming that the new wet well will be 8 foot diameter the required volume of storage between the pump on and pump off switch would be 3,745 gallons (500.6 $ft^3$). This equates to approximately 10 feet between the switches. To keep the inflow pipe from being submerged, the high water alarm should be set approximately 1 foot above the pump on switch and 1 foot below the invert into the wet well. It is also assumed that a minimum of 2 feet of depth will be maintained below the facility's storage volume to ensure the pump intakes are adequately covered. The existing lift station has a pipe inlet invert of approximately 5 feet below ground surface; using this inlet invert in the new facility the total depth of the wet well will be approximately 19 feet from ground surface. This configuration will provide adequate capacity within the wet well so that during peak flow periods the system would no longer surcharge back into the collection network and overflow into the nearby slough.

The existing above ground structure is poor condition therefore a new building will be required to house the new electrical and control equipment and backup generator. A building approximately 10 feet x 14 feet would be adequate

A preliminary cost estimate for this alternative is provided below:

**Table 7.2.2.2. Ammon Road Lift Station Upgrades – New Lift Station Cost Estimate**

| Ammon Road Lift Station – New Lift Station | | | | | |
|---|---|---|---|---|---|
| Item | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization, Insurance, Overhead, Bonds (10%) | LS | 1 | $110,389 | $110,389 |
| 2 | Construction Facilities/Temporary Systems/Bypass Provisions | LS | 1 | $35,000 | $35,000 |
| 3 | Wetw ell w ith Polyurea Coating, Excavation, Installation | LS | 1 | $210,000 | $210,000 |
| 4 | 100 HP Pump, VFD, Accessories and Installation | EA | 2 | $85,000 | $170,000 |
| 5 | Electrical, Wiring, Panels, Level Controls, SCADA | LS | 1 | $90,000 | $90,000 |
| 6 | Relocate Generator, Fuel Supply, ATS, Ventilation & Ducting | LS | 1 | $8,250 | $8,250 |
| 7 | Electrical & Generator Building w /Dividing Wall & Rollup Door | LS | 1 | $85,000 | $85,000 |
| 8 | Site Piping, Valves, Fittings and Vault | LS | 1 | $60,000 | $60,000 |
| 9 | 15-Inch Influent Pipe | LF | 20 | $286 | $5,720 |
| 10 | Site Work | LS | 1 | $20,000 | $20,000 |
| 11 | 10" Force main | LF | 20 | $195 | $3,900 |
| 12 | New Manhole | LF | 1 | $4,500 | $4,500 |
| 13 | Demolition and Abandonment of Lift Station | LS | 1 | $24,750 | $24,750 |
| 14 | Misc. Restoration and Clean Up | LS | 1 | $15,000 | $15,000 |
| | | Construction Total | | | $842,509 |
| | | Contingency (20%) | | | $168,502 |
| | | Subtotal | | | $1,011,011 |
| | | Engineering (20%) | | | $202,202 |
| | | Environmental Report | | | $20,000 |
| | | Environmental Engineering* | | | $40,000 |
| | | Administrative Costs (3%) | | | $30,330 |
| | | **Total Project Cost** | | | $1,303,543 |

*If needed

### 7.2.2.3. Ammon Road Lift Station - Force Main

The 10-inch force main for the Ammon Road lift station was constructed in 1999-2000 and constructed out of cement lined ductile iron pipe. The force main is routed up Sturdevant Road to 10th Street then down 10th Street to the wastewater treatment plant where it discharges. At 1400 gpm, the velocity in a 10-inch force main would be nearly 5.7 ft/s which is within the desirable limits defined by DEQ. Due to the age of the force main and the reasonable velocities no replacement or major modifications are suggested at this time for the existing Ammon Road Lift Station force main.

7.2.2.4. Ammon Road Lift Station - Summation and Recommendations

A number of improvements to the Ammon Road Lift Station have been discussed above. These included upgrading the station as a dry well station which is anticipated to be less expensive than constructing a new wet well style facility. With the current collection system configuration this is one of the two primary lift stations within the City that supply the majority of the flows to the wastewater treatment plant. Because of this system configuration this facility is a critical component of the wastewater system. It is also recognized that the City has indicated a desire to eliminate confined space entry requirements for the Ammon Road Lift Station. Therefore, it is recommended that the City undertake a project to replace the current wet/dry wells with a new wet well style lift station adjacent to the existing lift station site. This will provide the City with a new lift station at a critical location within the collection system capable of addressing current flows as well as future flows as the community continues to expand.

7.2.3. **High School Lift Station**

See section 4 for discussion on the current condition of the High School Lift Station. A minor improvement project could be complete to improve the operations and reliability of the High School Lift Station.

**Table 7.2.3. High School Lift Station Data**

| **High School Lift Station** | |
|---|---|
| Location | End of private drive off of Service Road |
| Type of Station | Wet well, duplex submersible |
| Pump Type | Non-clog, constant speed submersible pump |
| Motor Data | 23 Hp |
| Firm Capacity | Approximately 325 gpm |
| Overflow Point | Overflow is at the wet well, the elevation in unknown. |
| Overflow Discharge | Discharges to Olalla Slough. |
| Auxiliary Power | At the time of this report, the High School Lift Station does not have a dedicated, permanent backup generator, however the City is planning on moving a 94KW generator to the site for permanent backup power from a rebuilt water lift station. |
| Current Flows | Current PIF are approximately 21 gpm. |
| Projected Flow | The 20 year projected PIF is 28 gpm. |
| Projected Capacity | This pump station does not need to be replaced during the planning period. |

7.2.3.1. High School Lift Station – Do Nothing Option

As the existing station operates relatively well under the existing configuration, the City may be able to do only necessary maintenance to keep the station operational for many more years. By not undertaking a capital improvement project for the station, monies could be used for maintenance or improvements of other facilities.

7.2.3.2. High School Lift Station – Upgrades and Life Extension Improvements

While the station may not require immediate upgrades to satisfy capacity or major operational deficiencies, an upgrade during the planning period to extend the useful life of the station may be appropriate. The upgrade should include sealing of the wet well to minimize infiltration, installation of a bi-pass pump connection and a flow meter to monitor flows. The facility also needs an update to the system controls and the installation of an on-site automatic backup power generator within an enclosure. Although no major complaints were identified related to the long detention time this issue should be monitored closely. An improvement to the facility's ventilation system as well as the installation of an air injection system may be required at some point in the future. Any upgrade project for this facility should address the minor issues with the existing building to extend the life of the station and improve the operation and the reliability of the station. The following cost estimate is provided for this alternative:

**Table 7.2.3.2 - High School Lift Station Upgrades Cost Estimate**

| High School Lift Station Improvements - Life Extension Upgrade | | | | | |
|---|---|---|---|---|---|
| **Item** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | LS | 1 | $10,000.00 | $10,000.00 |
| 2 | Construction Facilities/Temporary Systems | LS | 1 | $4,200.00 | $4,200.00 |
| 3 | New station piping, valves, bypass, and fittings | LS | 1 | $12,000.00 | $12,000.00 |
| 4 | Electrical upgrades | LS | 1 | $32,000.00 | $32,000.00 |
| 5 | Onsite Backup Generator and Enclosure | LS | 1 | $40,000.00 | $40,000.00 |
| 6 | New controls, VFD's, and telemetry | LS | 1 | $16,000.00 | $16,000.00 |
| 7 | Concrete coating and repair in pits | LS | 1 | $12,000.00 | $12,000.00 |
| 8 | New Flow Meter and Manhole on Force Main | LS | 1 | $21,500.00 | $21,500.00 |
| 9 | Control/Electrical Building repair | LS | 1 | $10,600.00 | $10,600.00 |
| | | Construction Total | | | $158,300.00 |
| | | Contingency (20%) | | | $31,660.00 |
| | | Subtotal | | | $189,960.00 |
| | | Engineering (20%) | | | $37,992.00 |
| | | Administrative costs (3%) | | | $5,698.80 |
| | | **Total Project Costs** | | | **$233,650.80** |

7.2.3.3. High School Lift Station - Summation and Recommendations

The High School Lift Station is in relatively good condition, it is recommended that the City not take immediate action for upgrades to the station. However, plans should be made within the first half of the planning period to complete the upgrades to extend the life of the station throughout the planning period and beyond. It is recommended that the city install a flow meter prior to priority 1 design at this lift station to validate the calculated flows for this facility. A flow meter will also allow the City to monitor the flows at this facility to better determine the appropriate timing of a major facility upgrade.

### 7.2.4. **Lincoln Way Lift Station**

See section 4 for discussion on the current condition of the Lincoln Way Lift Station. A minor improvement project could be complete to improve the operations and reliability of the Lincoln Way Lift Station.

**Table 7.2.4 - Lincoln Way Lift Station Data**

| **Lincoln Way Lift Station** | |
|---|---|
| Location | Lincoln Way and Frontage Road |
| Type of Station | Wet well, duplex submersible |
| Pump Type | Non-clog, constant speed submersible pump |
| Motor Data | 30 Hp |
| Firm Capacity | Approximately 290 gpm |
| Overflow Point | Overflow is at manhole C-3, the elevation in unknown. |
| Overflow Discharge | Discharges to ditch at Frontage Road which drains to Depot Slough. |
| Auxiliary Power | Permanent 80 KW diesel generator. |
| Current Flows | Current PIF are approximately 313 gpm. |
| Projected Flow | The 20 year projected PIF is 361 gpm. |
| Projected Capacity | This pump station may need to be replaced/upgraded during the planning period. |

### 7.2.4.1. Lincoln Way Lift Station – Do Nothing Option

As the existing station operates relatively well under the existing configuration, the City may be able to do only necessary maintenance to keep the station operational for many more years. By not undertaking a major capital improvement project for the station, monies could be used for maintenance or improvements of other facilities. This option will mean at some point towards the end of the planning period the Lincoln Way Lift Station may not have the capacity to meet the design standard for lift stations. To address this concern a flow meter should be installed and monitored over the planning period to determine if the flow projections for this facility will be met. If the projected peak flow is actualized in the latter parts of the planning period a facility improvement project to address the lift station's capacity should be established and completed to extend the facility's operation life.

### 7.2.4.2. Lincoln Way Lift Station – Upgrades and Life Extension Improvements

While the station may not require immediate upgrades to satisfy current capacity or major operational deficiencies, an upgrade during the planning period would help to extend the useful life of the station and may be appropriate. The upgrade should include repairing the air injection system to ensure proper operation of that system. Installation of a bi-pass pumping connection and flow meter to monitor flows. The upgrades should also address the settling of the existing building and the installation/construction of a generator enclosure to extend the life of the station and improve the operation and reliability of the station.

The following cost estimate is provided for this alternative:

**Table 7.2.4.2. Lincoln Way Lift Station Upgrades Cost Estimate**

| Hospital Lift Station Improvements - Life Extension Upgrade | | | | | |
|---|---|---|---|---|---|
| **Item** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | LS | 1 | $10,000.00 | $10,000.00 |
| 2 | Construction Facilities/Temporary Systems | LS | 1 | $1,800.00 | $1,800.00 |
| 3 | New station piping, valves, bypass, and fittings | LS | 1 | $12,000.00 | $12,000.00 |
| 4 | Electrical upgrades | LS | 1 | $1,400.00 | $1,400.00 |
| 5 | Onsite Backup Generator and Enclosure | LS | 1 | $40,000.00 | $40,000.00 |
| 6 | Repair of Air Injection System | LS | 1 | $3,600.00 | $3,600.00 |
| 8 | New Flow Meter and Manhole on Force Main | LS | 1 | $21,500.00 | $21,500.00 |
| 9 | Repair and Stabilization of Control/Electrical Building | LS | 1 | $10,600.00 | $10,600.00 |
| | | Construction Total | | | $100,900.00 |
| | | Contingency (20%) | | | $20,180.00 |
| | | Subtotal | | | $121,080.00 |
| | | Engineering (20%) | | | $24,216.00 |
| | | Administrative costs (3%) | | | $3,632.40 |
| | | **Total Project Costs** | | | **$148,928.40** |

### 7.2.4.3. Lincoln Way Lift Station - Summation and Recommendations

Because the Lincoln Way Lift Station is in relatively good condition, it is recommended that the City not take immediate action for upgrades to the station. However, it is recommended that a flow meter be installed at the facility to verify and monitor flows at the facility. The installation of a flow meter will assist in determining if actual facility flows at this point in time dictate the need to upgrade the capacity of the lift station. It is also recommended that plans be made within the first half of the planning period to complete the upgrades identified to extend the life of the station throughout the planning period and beyond.

### 7.2.5. **Butler Bridge Lift Station**

See section 4 for discussion on the current condition of the Butler Bridge Lift Station. A significant improvement project will be required at the Butler Bridge Lift Station in order to address the existing problems.

Investigations into the current and projected flows for this lift station have resulted in the following peak instantaneous flows that this facility must be capable of handling:

Current Peak Instantaneous Flow ............................................................. 4.51 mgpd (3,132 gpm)
Projected Peak Instantaneous Flow ........................................................... 5.23 mgpd (3,632 gpm)

As stated in Section 6, lift stations must be designed to handle the peak instantaneous flows.

Therefore, based on this analysis, the Butler Bridge Lift Station needs be able to handle a projected firm pumping capacity of 3,632 gpm. This can be accomplished with approximately two 3,650 gpm pumps (duplex) or three 1,820 gpm pumps (triplex).

The recommended wet well volume for this facility is defined by two basic criteria. The first, the facility must be designed to prevent excessive number of pump starts per hour. Pump manufacturers typically recommend a maximum of 15 starts per hour and designing for approximately 10 starts per hour. For constant speed pumps, the minimum wet well volume between low water level (LWL) and pump on level can be calculated using the following formula:

$$V_{minimum} = (T_{minutes} \text{ x } Q_{max}) / 4$$

$V_{minimum}$ = Minimum volume in cubic feet
$T_{minutes}$ = Target time between pump starts in minutes (10 starts per hour or 6 minutes)
$Q_{max}$ = Pump design capacity, use 3,650 gpm (487.9 $ft^3$/minute)

Therefore: $V_{minimum} = (\text{ 6 minutes x } 487.9 \text{ ft}^3\text{/minute}) / 4 = 731.9 \text{ ft}^3$ (5,475 Gallons)

The second criteria used to define wet well volume identify the maximum storage volume allowed while avoiding septic conditions within the wet well. In general, average detention time should be no more than 35 minutes during average flow conditions during the dry season. The average maximum wet well volume required to avoid septic conditions can be calculated as follows:

$$V_{wetwell} = Q_{summer} \text{ x 35 minutes}$$

$V_{wetwell}$ = Maximum wetwell volume to avoid septic conditions
$Q_{summer}$ = Dry season average flow (Approximate) = 275 gpm

Therefore: $V_{wetwell}$ = 275 gpm x 35 minutes = 9,625 gallons (1,286.7 $ft^3$)

Based on these calculations a properly sized wet well for the A Street lift station should have a minimum wet well storage volume of 5,475 gallons and a maximum storage volume of 9,625 gallons. These limits will prevent excessive pump starts which can increase the wear on the pump stations pumps as well as limit the detention time preventing the development of septic conditions within the wet well.

To address the deficiencies at this lift station improvement alternatives were developed and are discussed below for the A Street Lift Station. A “do nothing” alternative will likely result in untreated wastewater overflows due to the significant flow and storage deficiency and the poor condition of the above ground structure.

**Table 7.2.5 – Butler Bridge Lift Station Data**

| Butler Bridge Lift Station | |
|---|---|
| Location | Butler Bridge Road, 1 mile north of Bridge |
| Type of Station | Wet well / dry well, duplex flooded suction |
| Pump Type | Vertically mounted, solids handling non-clog centrifugal pump with a VFD |
| Motor Data | 100 Hp |
| Firm Capacity | Approximately 2,160 gpm at 108’ TDH |
| Overflow Point | Overflow is at manhole J-1, the elevation in unknown. |
| Overflow Discharge | Discharges to Depot Slough. |
| Auxiliary Power | On-Site automatic transfer switch 100 KW diesel generator with 50 gallon fuel capacity. |
| Current Flows | Current PIF are approximately 3,132 gpm. |
| Projected Flow | The 20 year projected PIF is 3,632 gpm. |
| Projected Capacity | This pump station is undersized and needs to be replaced during the planning period. |

#### 7.2.5.1. Butler Bridge Lift Station – Dry well Upgrade

Because the existing station is a dry well/wet well type station, capacity to the station could be increased through the installation of new pumps in the dry well. It is becoming increasingly common to install submersible solids handling pumps in a dry well configuration. This provides the advantages of submersible solids handling capabilities and reliabilities with the ease of installation of a dry well pump.

The disadvantages of continuing to operate the station as a dry well lift station are numerous. Firstly, the deep dry and wet wells are considered confined spaces which necessitate special safety measures for anyone entering the pits. Harnesses, hoists, ventilation, gas detection, multiple personnel, and other considerations must be met before anyone can enter the pits to perform maintenance or observe the operation of the pumps.

This facility was originally constructed over 50 years ago and it has had multiple updates over its life many of the original components are still being used today. Some of these components and some of the updated components are showing their age and could require replacement. Some of these components include some of the piping and fittings, valves, hooks, tie-offs, access ladders, electrical systems, control systems, wet and dry wells, and the above ground control building and the backup generator enclosure.

A significant disadvantage to continuing to operate the station as a dry well/wet well station is the limitations in the wet well capacity. The existing wet well can hold 284 gal per foot of depth; at the 3 foot range from the existing pump on-pump off switches the well has a storage capacity of 853 gallons (114.0 $ft^3$). As defined above this facility should provide a minimum of 5,475 gallons of storage which is significantly more than the current wet well capacity. This deficiency accelerates wear on the pumps, increasing the maintenance and repairs required over the useful life of the facility.

If this alternative is selected, the City must install two new pumps, each capable of 3,650 gpm. While lower flows can be addressed using VFD’s, the small wet well will result in less operating flexibility and more starts and stops on the pumps.

A preliminary cost estimate is provided below for the dry well upgrade alternative:

**Table 7.2.5.1 - Butler Bridge Lift Station Upgrades – Dry well Upgrade Cost Estimate**

| Butler Bridge Pump Station – Dry Pit Upgrade | | | | | |
|---|---|---|---|---|---|
| **Item** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | LS | 1 | $41,000 | $41,000 |
| 2 | Construction Facilities/Temporary Systems/Demolition | LS | 1 | $25,000 | $25,000 |
| 3 | Bypass pumping | LS | 1 | $11,500 | $11,500 |
| 4 | 100 HP Pump, VFD, Accessories and Installation | EA | 2 | $85,000 | $170,000 |
| 5 | Electrical, Wiring, Panels, Level Controls, SCADA upgrades | LS | 1 | $90,000 | $90,000 |
| 6 | New Site Piping and Fittings (re-use existing valves) | LS | 1 | $28,000 | $28,000 |
| 7 | Concrete coating and repair in pits | LS | 1 | $17,000 | $17,000 |
| 8 | Control and Generator Building improvements | LS | 1 | $22,500 | $22,500 |
| 9 | Misc. Restoration and Clean Up | LS | 1 | $2,350 | $2,350 |
| | | Construction Total | | | $407,350 |
| | | Contingency (20%) | | | $81,470 |
| | | Subtotal | | | $488,820 |
| | | Engineering (20%) | | | $97,764 |
| | | Administrative Costs (3%) | | | $14,665 |
| | | **Total Project Cost** | | | **$601,249** |

7.2.5.2. Butler Bridge Lift Station – New Wet Well

The City has indicated a desire to eliminate the confined space and explosion issues related to the current wet well/dry well station. The simplest way to accomplish this is to construct a new pump station wet well adjacent to the existing pump station and install new submersible pumps in the wet well.

Construction of a new wet well adjacent to and between the existing station and the RV discharge facility should not require acquisition of additional property. Figure 7.2.5.2 shows the approximate configuration and layout of the new facility as it relates to the existing site features.

The new wet well could be set up as a tri-plex wet well to provide greater operational flexibility. This would allow the City to install two pumps now (each capable of the firm pumping capacity of 3,650 gpm) and adding a third in the future should the need arise. A current option would be to install three smaller pumps (1,820 gpm each) to meet the capacity and redundancy requirements. A triplex configuration would be better able to accommodate potential increases in flow beyond the 20-year planning period.

As identified previously, this facility should provide a minimum storage capacity of 5,475 gallons and a maximum storage capacity of 9,625 gallons. By selecting to use the 9,625 gallons storage capacity this facility has the ability to adequately address current as well as future flows while helping to minimize the chance for an overflow event. Assuming that the new wet well will be 10 foot in diameter the required depth between the pump on and pump off switch would need to be approximately 17 feet between the switches. To prevent a backup into the collection piping the high water alarm should be set approximately 1 foot above the pump on switch and 1 foot below the invert into the wet well. It is also assumed that a minimum of 2 feet of depth will be maintained below the facility's storage volume to ensure the pump intakes are adequately covered. The existing lift station has an invert of approximately 3.5 feet below ground surface; using this as the invert in the new facility the total depth of the wet well will be approximately 24.5 feet from ground surface. This configuration will provide adequate capacity within the wet well so that during peak flow periods the system would no longer surcharge back into the collection network and overflow into the nearby slough.


Civil West Engineering Services, Inc.

Dwg By: MDW
Date: FEB 25, 2011

0 1"

**Butler Bridge Lift Station Proposed Layout**

FIGURE 7.2.5.2

WASTE WATER FACILITY PLAN

CITY OF TOLEDO
Lincoln County, Oregon

The existing above ground structure is poor condition therefore a new building will be required to house the new electrical and control equipment and backup generator. A building approximately 10 feet x 14 feet would be adequate.

A preliminary cost estimate for this alternative is provided below:

**Table 7.2.5.2b - Butler Bridge Lift Station Upgrades – New Lift Station Cost Estimate**

| Butler Bridge Pump Station - All New | | | | | |
|---|---|---|---|---|---|
| **Item** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Mobilization, Insurance, Overhead, Bonds (10%) | LS | 1 | $91,109 | $91,109 |
| 2 | Construction Facilities, Temporary Systems and Bypass Provisions | LS | 1 | $35,000 | $35,000 |
| 3 | Wetw ell w ith Polyurea Coating, Excavation, Installation | LS | 1 | $210,000 | $210,000 |
| 4 | 100 HP Pump, VFD, Accessories and Installation | EA | 3 | $85,000 | $255,000 |
| 5 | Electrical, Wiring, Panels, Level Controls, SCADA | LS | 1 | $90,000 | $90,000 |
| 6 | Relocate 100 kW Generator, Fuel Supply, ATS, Ventilation and Ducting | LS | 1 | $8,250 | $8,250 |
| 7 | Electrical &Generator Building, 240 sq ft, w /Dividing Wall & Rollup Door | LS | 1 | $85,000 | $85,000 |
| 8 | Site Piping, Valves, Fittings and Vault | LS | 1 | $60,000 | $60,000 |
| 9 | 18-Inch Influent Pipe | LF | 20 | $300 | $6,000 |
| 10 | Site Work | LS | 1 | $20,000 | $20,000 |
| 11 | 14" Force main | LF | 20 | $324 | $6,480 |
| 12 | New Manhole | LF | 1 | $4,500 | $4,500 |
| 13 | Demolition and Abandonment of Lift Station | LS | 1 | $24,750 | $24,750 |
| 14 | Misc. Restoration and Clean Up | LS | 1 | $15,000 | $15,000 |
| | | Construction Total | | | $911,089 |
| | | Contingency (20%) | | | $182,218 |
| | | Subtotal | | | $1,093,307 |
| | | Engineering (20%) | | | $218,661 |
| | | Environmental Report | | | $20,000 |
| | | Environmental Engineering* | | | $40,000 |
| | | Administrative Costs (3%) | | | $32,799 |
| | | **Total Project Cost** | | | **$1,404,767** |

*If needed

### 7.2.5.3. Butler Bridge Lift Station - Summation and Recommendations

A number of alternatives for improvements to the Butler Bridge Lift Station have been discussed above. These included upgrading the station as a dry well station as well as constructing a new wetwell station with submersible pumps. While the dry well option is anticipated to be less expensive, this facility is one of two that supply a majority of the flows to the treatment plant making this facility critical to the operation of the City's wastewater system. This critical nature coupled with the City's desire to eliminate confined space entry requirements establishes this facility as a primary focus to improve capacity and operational reliability. Therefore, it is recommended that the City undertake a project to install a new wetwell with a new control building at the site to provide a submersible pumping station. This will provide the City with a modern pump station, eliminate confined space entry issues, and allow the City to expand the station in the future with reduced expenses.

### 7.2.5.4. Butler Bridge Lift Station - Force Main

The 14-inch force main for the Butler Bridge lift station was originally constructed in 1982. In 2010 a section of the force main was replaced. As it stands today, the force main is a combination of ductile iron and HDPE pipe. The force main is routed down Butler Bridge Road to a point where it connects to the new HDPE pipe and then passes under the existing train tracks. The force main then heads uphill to the

WWTP headworks. Although the older section of the pipe is relatively young (~30 years) it has recently been prone to breaks. This may be due to the high traffic volume which crosses the pipeline to access the mill. Regardless of the reason, it is recommended to replace the old section of the pipe with HDPE.

At 3,650 gpm, the velocity in a 14-inch force main would be nearly 10 ft/s which is marginally higher than the DEQ recommended velocity for a force main. Therefore, a new 16-inch force main is recommended which will have a velocity of just above 7.5 ft/s at a flow rate of 3650 gpm.

Because the force main is between Butler Bridge Road and the GP property fence it should be a good candidate for open trench construction. A preliminary cost estimate for this alternative is provided below:

**Table 7.2.5.3 – Butler Bridge Force Main – Open Trench Construction Cost Estimate**

| A Street Lift Station - New 12-Inch Force Main | | | | | |
|---|---|---|---|---|---|
| **Item** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Mobilization, Insurance, Overhead, Bonds (10%) | LS | 1 | $16,140.0 | $16,140 |
| 2 | Construction Facilities, Temporary Systems and Bypass Provisions | LS | 1 | $23,000 | $23,000 |
| 3 | New 14-Inch HDPE Force Main | LF | 1100 | $120 | $132,000 |
| 4 | Tie ins, Fittings, etc. | ea | 1 | $6,400 | $6,400 |
| | | Construction Total | | | $177,540 |
| | | Contingency (20%) | | | $35,508 |
| | | Subtotal | | | $213,048 |
| | | Engineering (20%) | | | $42,610 |
| | | Administrative Costs (3%) | | | $6,391 |
| | | **Total Project Cost** | | | **$262,049** |

## *7.3. WWTP*

As discussed in section 4.3, because of the lack of sufficient biosolids storage capacity one of the two treatment units is regularly off-line. The implementation of the recommendations identified in this section will allow the year-round use of both treatment units which will considerably increase the treatment capacity of the plant.

### 7.3.1. **Headworks**

The existing headworks are appropriately sized to handle the current expected peak flow (6.5 MGD). As calculated in section 5.1.6 the 20 year projected flows are larger and may overwhelm the headworks. The projected flow is based on population growth but does not account for recent and planned I/I improvements. The recent and future I/I repair work will likely decrease peak flows, although the amount of reduction is unknown. To be conservative in design of the facilities, it is assumed that there will be no reduction. It is likely that the projected peak flows will not be realized even if the expected growth occurs. It is recommended that the headworks not be enlarged at this point, although should significant development occur, this may be required at that time.

Per the 1993 construction documents the surge vault was designed to equalize low flows to provide a consistent flow rate into the treatment units. A steady flow rate increases the efficiency in the activated sludge treatment process. Since the construction, the floating weir mechanism which facilitates the low flow equalization was removed from the surge tank due to the inoperability of the unit. We suggest that a new unit be designed and installed to equalize low flows. This is not critical, but will help maintain consistent treatment efficiencies. The estimated cost to re-design and replace this mechanism is $25,000.

### 7.3.2. **WWTP – Outfall Improvements**

As presented in section 4.3.6, the existing effluent outfall is hydraulically incapable of discharging high flows during high tides. The headwater elevation of the discharge is the flowmeter vault, which, according to the 1993 construction plans, has a WSEL of approximately 12.4’ ASL. The water level in the Yaquina River averages approximately 3.5’ ASL, however, high tides will occasionally result in water levels over 10’ ASL, leaving less than 2.5 feet of head available to discharge effluent flows. Even a bank-side outfall would not be able to convey peak flows during high tide. At approximately 1500 feet in length, the 18” outfall pipe induces almost 10’ of head loss at a peak flow of 6.5 MGD.

#### 7.3.2.1. Outfall Pipe

During a previous investigation, it was noted that the section of the discharge pipe between the treatment plant and the old drying beds was in very poor condition, with sections of the pipe broken and mis-aligned. Gary Utiger, the WWTP operator, has also indicated that there is a broken end of a cleaning jet in this section of pipeline. For these reasons it is recommended that the City replace at least the northernmost 300 feet of the 18” effluent pipe.

A preliminary cost estimate is provided below for the section of effluent pipe replacement:

**Table 7.3.2.1 WWTP – Outfall Pipe Cost Estimate**

| Outfall Pipe Replacement | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $14,600.00 | $14,600.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $8,800.00 | $8,800.00 |
| 3 | Directional Drill 24-inch HDPE Pipe | lf | 300 | $370.00 | $111,000.00 |
| 4 | Connect to Existing | ls | 1 | $6,000.00 | $6,000.00 |
| | | Construction Total | | | $140,400.00 |
| | | Contingency (20%) | | | $28,080.00 |
| | | Subtotal | | | $168,480.00 |
| | | Engineering (20%) | | | $33,696.00 |
| | | Administrative costs (3%) | | | $5,054.40 |
| | | **Total Project Costs** | | | **$207,230.40** |

#### 7.3.2.2. Effluent Booster Pumps

Because of the minimal head available during high tide and storm events, we recommend that the city install low pressure, high volume propeller pumps capable of pumping the PIF. The pumps would be installed in the downstream side of the effluent meter structure. This will need to be reconstructed to accept the pumps.

Because the original outfall pipe is currently operated in a gravity drain scenario, there are concerns that this pipe will be able to withstand even the minor pressure increase generated by the propeller pumps. The outfall pipe replacement identified above should be constructed prior to, or at the same time as the effluent pumps.

A preliminary cost estimate is provided below for the effluent pumps:

**Table 7.3.2.2 WWTP - Effluent Booster Pumps Cost Estimate**

| Effluent Booster Pumps | | | | | |
|---|---|---|---|---|---|
| **Item** | **Description** | **Units** | **Quantity** | **Unit Cost** | **Total Cost** |
| 1 | Mobilization, Insurance, Overhead, Bonds (8%) | LS | 1 | $12,000 | $12,000 |
| 2 | Construction Facilities, Temporary Systems & Bypass Provisions (6%) | LS | 1 | $9,000 | $9,000 |
| 3 | Rebuild Effluent Sump | LS | 1 | $15,000 | $15,000 |
| 4 | 15 HP Pump, Accessories and Installation | EA | 2 | $35,000 | $70,000 |
| 5 | Electrical, Wiring, Panels, Level Controls, SCADA | LS | 1 | $22,000 | $22,000 |
| 8 | Site Piping, Valves, Fittings and Vault | LS | 1 | $30,000 | $30,000 |
| 10 | Site Work | LS | 1 | $5,500 | $5,500 |
| 14 | Misc. Restoration and Clean Up | LS | 1 | $3,800 | $3,800 |
| | | Construction Total | | | $167,300 |
| | | Contingency (20%) | | | $33,460 |
| | | Subtotal | | | $200,760 |
| | | Engineering (20%) | | | $40,152 |
| | | Administrative Costs (3%) | | | $6,023 |
| | | **Total Project Cost** | | | **$246,935** |

### 7.3.3. WWTP - Biosolids Management

The City of Toledo has an existing Biosolids Management Plan (Appendix D) which has been reviewed and approved by DEQ. Presently, WAS and RAS sludge are removed from liquid stream after settling into a hopper located at the bottom of the TU1 and TU2 clarifiers. The RAS is pumped back into the TU1 and TU2 aerators. The WAS is pumped into the facility's digesters and allowed to decompose under complete mix aerobic conditions. After the biosolid digestion is complete it is stored in a 92,000 gallon biosolids storage tank. The digested sludge is stored at the treatment plant through the winter season then during the summer season it is transferred to a tanker truck which hauls the sludge to an offsite facility for land application.

As discussed in Section 4 of this report the current sludge/biosolids storage capacity is inadequate during the winter months, reducing the treatment plant total treatment capacity due to the need to use TU1 aerator, clarifier, and digester as a sludge storage facility. To address this issue below is an investigation into alternative ways to address the sludge storage issues at the Toledo waste water treatment plant.

Current digester capacity of the wastewater treatment plant consists of the TU1 digester (114,000 gallons), TU2 digester (65,000 gallons), and a 200,000 gallon digester constructed as in the 2000/2001 improvement project. These three facilities provide a total aerobic digester capacity of approximately 379,000 gallons. According to treatment plant staff, in addition to these storage facilities the operator(s) will also take the TU1 aerator (116 ,000 gallons) and the TU1 clarifier (140,000 gallons) off line and use them to store sludge during the winter months. The operator(s) did indicate that the 140,000 gallon clarifier typically does not use more than half of its storage capacity while all the other facilities typically are filled to their full capacity. In addition to the above storage facilities the WWTP has a 92,000 gallon biosolids storage tank. When all of the above facilities are used the approximate total volume of sludge/biosolids that is stored onsite is 657,000 gallons. This additional volume requirement may be reduced by installing covers on some, or all, of the existing open air digesters to reduce the volume of rain water entering the treatment system.

#### 7.3.3.1. Sludge Storage, Alternative 'A'

To maintain the full treatment capacity through the entire year TU1 must be available for secondary treatment. This alternative includes the construction of a 190,000 gallon (min) sludge/biosolids storage facility. Property is available on the north, up-hill side, of the treatment plant property. Locating a storage facility in this location would require that the sludge be pumped from the lower end of the plant.

A preliminary cost estimate has been prepared for this alterative and is available below. This estimate includes the installation of a 190,000 gallon glass fused-to-steel tank, site work, all the piping and valves required to connect it to the existing facilities as well as sludge pump to fill the tank:

**Table 7.3.3.1 WWTP – Sludge Storage Alternative 'A' Cost Estimate**

| 190,000 Gallon Biosolids Storage Tank Improvements | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $50,000.00 | $50,000.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $7,800.00 | $7,800.00 |
| 3 | 190k gallon Storage Tank (Glass fused-to-steel) | ls | 1 | $210,000.00 | $210,000.00 |
| 4 | Sludge pump and controls | ls | 1 | $15,500.00 | $15,500.00 |
| 5 | Piping, fittings,, valves, and vaults | ls | 1 | $43,500.00 | $43,500.00 |
| 6 | Site w ork, fencing, paving, flatw ork | ls | 1 | $22,000.00 | $22,000.00 |
| | | Construction Total | | | $348,800.00 |
| | | Contingency (20%) | | | $69,760.00 |
| | | Subtotal | | | $418,560.00 |
| | | Engineering (20%) | | | $83,712.00 |
| | | Administrative costs (3%) | | | $12,556.80 |
| | | **Total Project Costs** | | | **$514,828.80** |

7.3.3.2. Sludge Thinkening, Alternative 'B'

As an alternative to increasing the storage capacity the City can decrease the volume of sludge/biosolids by thickening the sludge/biosolids. To ensure that the treatment plant's capacity is not reduced through the winter months the facility needs to reduce the total volume of sludge that it needs to store to approximately 471,000 gallons. To do this a means of dewatering the sludge needs to be incorporated at the end of the treatment process and prior to the biosolids storage tank. According to the Biosolids Management Plan the City of Toledo land applies its biosolids during the dry season. The average percent solids of the biosolids that it land applied is 3.32%. If a dewatering system capable of increasing the percent solids from 3.32% to 6% is installed at the end of the treatment train a substantial decrease would be seen in the total volume that would need to be stored. The amount of storage after the primary treatment processes currently consists of a 92,000 gallon biosolids storage tank. This tank currently stores 92,000 gallons of 3.32% solids biosolids. If the sludge was thickened to 6%, this tank could store an equivalent volume of 166,000 gallons. The remaining sludge volume of 491,000 gallons would need to be stored in the facilities digesters. The available digester storage volume is 379,000 gallons which is not adequate to meet the storage needs of the treatment plant.

According to the plant operator(s) the flow pattern through the plant for the sludge begins in TU2 digester at 1.0% solids. From this digester it routs to the 200,000 gallon digester where the percent solids is increased to 2.0%. The sludge then flows to the TU1 digester where the solids are increased to 2.5% before it is sent to the biosolids storage tank where the final percent solids, 3.32% is achieved. A solution to the storage issue could be achieved by taking TU1 digester out of the treatment process and reclassifying it as a biosolids storage tank. To determine if TU1 can be removed first the residence time for the sludge must be checked to ensure the treatment process still meets design requirements.

Requirements for achieving class B biosolids indicate that the mean cell residence time and temperature must be forty days at 20°C (68°F) or sixty days at 15°C (59°F). According to the Biosolids Management

Plan the facility land applies 258,000 gallons of 3.32% solids sludge per year. This gives us an approximate daily sludge production rate of 2,400 gallons assuming 1.0% is the initial percent solids. Using this production rate the table below identifies the current residence time in each digestion facility:

**Table 7.3.3.2a - Sludge Residence Time**

| Digester | Capacity, Gallons | Reported % Solids | Sludge Residence, Days |
|---|---|---|---|
| TU2 | 65,000 | 1.00% | 27 |
| 200K | 200,000 | 2.00% | 83 |
| TU1 | 114,000 | 2.50% | 47 |
| Biosolids Tank | 92,000 | 3.32% | 38 |

Through the first two digesters the facility has an approximate residence time of 110 days. According to the Biosolids Management Plan the operation temperatures for the digesters and holding tanks range from a low of 12.2°C to a high of 21.2°C. Using this information and assuming the coldest temperature of 12°C the sludge residency time must be a minimum of 72 days, which is significantly less than the 111 days provided by the TU2 and 200,000 gallon digesters.

With the Class B biosolids requirements being meet with the TU2 and 200,000 gallon digesters the TU1 digester could be converted to an additional biosolids storage reservoir. According to the information available the TU1 currently holds 114,000 gallons of 2.5% solids sludge; if the sludge was thickened to 6% this tank could store and equivalent volume of 273,600 gallons. The remaining sludge volume of 217,400 gallons would need to be stored in the facilities 200,000 gallon and TU2 digesters, which have an available capacity of 265,000 gallons. The available digester storage volume is more than adequate to address the remaining sludge volume while maintaining both treatment lines open throughout the year.

In summary sludge Alternative ‘B’ would install a sludge thickener between the 200,000 gallon digester and TU1 digester. The TU1 digester and the existing biosolids storage tank would become dedicated biosolid storage facilities. This configuration would reduce the likelihood that the plant would need to use one of its treatment trains as a sludge storage facility improving the plants ability to treat peak flow events throughout the year. One additional improvement that may be required with the installation of a sludge thickener is either a modification or replacement of the current tanker truck used for land application of the facilities sludge to be able to distribute a thicker sludge. A preliminary investigation into the cost to address the tanker truck concerns has been completed and is provided along with this alternative’s preliminary cost estimate.

A preliminary cost estimate is provided below for the sludge storage alternative:

**Table 7.3.3.2b - WWTP – Sludge Thickening Alternative Cost Estimate**

| WWTP Sludge Thickener Improvements | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Bonds, Insurance, Overhead, Mobilization Costs | ls | 1 | $35,000.00 | $35,000.00 |
| 2 | Construction Facilities/Temporary Systems | ls | 1 | $7,800.00 | $7,800.00 |
| 3 | Sludge Thickening Building | sf | 100 | $265.00 | $26,500.00 |
| 4 | 25 gpm Rotary Screen Thickener w/Flocculation System and NEMA 4X Control Panel (Skid Mounted) | ls | 1 | $85,000.00 | $85,000.00 |
| 5 | Piping, Fittings, Valves, and Vaults | ls | 1 | $23,500.00 | $23,500.00 |
| 6 | TWAS Pump with VFD | ls | 1 | $65,000.00 | $65,000.00 |
| 7 | Site Work, Fencing, Paving, Flatwork | ls | 0 | $11,749.00 | $0.00 |
| 8 | Replacement Sludge Field Spreader | ls | 1 | $110,000.00 | $110,000.00 |
| | | Construction Total | | | $352,800.00 |
| | | Contingency (20%) | | | $70,560.00 |
| | | Subtotal | | | $423,360.00 |
| | | Engineering (20%) | | | $84,672.00 |
| | | Administrative costs (3%) | | | $12,700.80 |
| | | **Total Project Costs** | | | **$520,732.80** |

7.3.3.3. Selection

Costs of the two options are relatively equal. Concern has been raised about the viability of land applying solids as high as 6%, specifically regarding the ability of the receiving property to absorb such a high solids content. Adding storage will not alter the current process and should be easier for operators to monitor. Because of these considerations our recommendation is that the City plan on adding a new sludge storage tank.

## *7.4. Alternatives Summary*

The tables below provide a concise summarization of the proposed improvements identified in Sections 7.1 through 7.3 of this report.

**Table 7.4a Collection System – Expansion Summary**

| Summary of Collection System Expansion to Service UGB: | | | |
|---|---|---|---|
| Service Area | Area Description | Service Type | Total Cost |
| 1 | Airport Peninsula | Gravity Collection | $816,317 |
| 1 | Airport Peninsula | Lift Station and Force Main | $1,064,732 |
| 2 | Southern Yaquina River | Gravity Collection | $514,644 |
| 2 | Southern Yaquina River | Lift Station and Force Main | $931,892 |
| 3 | Southern Sturdevant Road | Gravity Collection | $1,280,246 |
| 4 | Central Sturdevant Road | Gravity Collection | $635,514 |
| 4 | Central Sturdevant Road | Lift Station and Force Main | $898,682 |
| 5 | Northern Olalla Slough | Gravity Collection | $1,786,956 |
| 5 | Northern Olalla Slough | Lift Station and Force Main | $1,207,018 |
| 6 | Hwy 20 | Gravity Collection | $1,041,835 |
| 7 | Sawmill | Gravity Collection | $309,237 |
| 8 | Sawmill Ponds | None | - |
| 9 | Southeast Ridge Line | None | - |
| 10 | High School | None | - |

**Table 7.4b Collection System – Improvement Alternatives**

| Summary of Collection System Improvements and Alternatives: | | | |
|---|---|---|---|
| **Facility** | **Alternative, Recommendation or Priority** | **Description** | **Total Cost** |
| Collection System (Piping and Manholes) | I & I - Priority 1 | Pipe Replacement, Lining, Bursting or Patching; Manhole Rehabilitation | $380,935 |
| | I & I - Priority 2 | | $565,400 |
| | I & I - Priority 3 | | $350,260 |
| | I & I - Priority 4 | | $140,080 |
| "A" Street Lift Station | Alternative A | Dry Pit Upgrade | $671,248 |
| | Alternative B | New Wet Well | $1,326,480 |
| "A" Street Lift Station Force Main | Recommendation | Replace Force Main | $172,175 |
| Ammon Road Lift Station | Alternative A | Dry Pit Upgrade | $621,064 |
| | Alternative B | New Wet Well | $1,303,543 |
| Ammon Road Lift Station Force Main | Recommendation | Do Nothing Option | - |
| High School Lift Station | Alternative A | Do Nothing Option | - |
| | Alternative B | Upgrades and Life Extension Improvements | $233,651 |
| High School Lift Station Force Main | Recommendation | Do Nothing Option | - |
| Hospital Lift Station | Alternative A | Do Nothing Option | - |
| | Alternative B | Upgrades and Life Extension Improvements | $148,928 |
| Hospital Lift Station Force Main | Recommendation | Do Nothing Option | - |
| Butler Bridge Lift Station | Alternative A | Dry Pit Upgrade | $601,249 |
| | Alternative B | New Wet Well | $1,404,767 |
| Butler Bridge Lift Station Force Main | Recommendation | Do Nothing Option | - |

## Table 7.4c WWTP – Improvement Summary

| Summary of WWTP System Improvements/Alternatives: | | | |
|---|---|---|---|
| **Facility** | **Alternative** | **Description** | **Total Cost** |
| Headworks | Recommendation | Replace Flow Equalization Wier | $25,000 |
| Outfall Pipe Replacement | Recommendation | Replace 300 lf | $207,230 |
| Effluent Booster Pumps | Recommendation | Install Effluent Booster Pumps | $246,935 |
| Biosolids Management | A | Consturct Additional Storage Tank | $514,829 |
| | B | Sludge Thicking Facility | $520,733 |

Section

8

# 8.0 Rate Study

This section of the Facilities Plan provides a comparison of costs of the various treatment process and collection system improvement alternatives developed in Section 7. Funding options expected to be available to the City of Toledo also are summarized herein.

In order for the City to plan for repayment of loans obtained in conjunction with the improvements, a method of determining the cost per user is required. A recent Water Rate Study was completed in January, 2012 by Civil West Engineering Services, Inc. for the City where EDUs coupled with the size of the water service serving a property was used to calculate water system user fees. This information will be utilized rather than existing sewer account information to determine the future rate structure required.

## 8.1. Estimated Annual Operation, Maintenance and Replacement Costs of the Proposed System

Multiple upgrades to the City of Toledo wastewater collection and treatment system were considered. Based on cost information presented in Section 7 and the operation, maintenance and replacement costs for each alternative an increase to the current wastewater rate is anticipated. In order to calculate the impact on rate payers it is important to understand the current user rate structure.

### 8.1.1. Current User Rates

Sewer system user rates in Toledo are based on the water meter size and the volume of water purchased by the customer as read on the water meter. Present sewer user rates for a standard residential or small commercial customer consists of a flat rate of $11.20 per month for first thousand gallons plus $14.83 per one thousand gallons of treated water based on the average amount of water that customer used during the months of January through April. Every May the utility department refigures each customer's average usage.

The average water usage in the city is 4,365 gallons per month during the winter months identified above. This results in an average sewer bill to wastewater customers of $61.00 per month.

### 8.1.2. Existing Sewer System Operating Budget

The City of Toledo Sewer Fund includes all revenue and expenses related to operation and maintenance of the existing wastewater collection and treatment system. The fund includes revenue collected from users in the form of monthly user fees and sewer connection charges. Operating expenses generally include personnel expenses, materials and services expenses, capital expenses, operating contingency, and loan repayment. The City also has established a Sewer Reserve Fund which is funded by transfers from the Sewer Fund. The following table presents the total or adopted revenue and expenses over the past three years and provides the adopted budget for the 2012 fiscal year.

**Table 8.1.2 Sewer Fund Revenue and Expense Summary**

| Fiscal Year | 2009 | 2010 | (Adopted) 2011 | (Adopted) 2012 |
|---|---|---|---|---|
| **Total Revenue** | **$805,060.69** | **$936,539.89** | **$1,006,520.00** | **$980,209.00** |
| Transfers | ($242,201.81) | ($348,772.57) | ($443,308.00) | ($403,957.00) |
| Sewer Loan Payment | ($145,352.00) | ($145,352.00) | ($145,352.00) | ($145,352.00) |
| Personnel Services | ($132,573.13) | ($136,864.48) | ($139,960.00) | ($149,220.00) |
| Marerials & Services | ($161,478.20) | ($170,210.65) | ($207,900.00) | ($211,680.00) |
| Contingency | $0.00 | $0.00 | ($70,000.00) | ($70,000.00) |
| **Total Expenditures** | **($681,605.14)** | **($801,199.70)** | **($1,006,520.00)** | **($980,209.00)** |
| **Overall Balance** | **$123,455.55** | **$135,340.19** | **$0.00** | **$0.00** |

As indicated in the above table, the City of Toledo has an existing debt loan payment of $145,352 per year for repayment of general sewer loan which was used to fund a previous sewer improvement project.

### 8.1.3. Reserve Funds

As mentioned in the previous section, the City has established a Sewer Reserve Fund with money transferred from the Sewer Fund annually. These fund acts as a savings account that will help finance the wastewater treatment and collection system improvements recommended in this Plan. According to the City's financial Statements, dated June 30, 2011 the balance of these funds is indicated in the following table.

**Table 8.1.3 Current Balances of Reserve Funds**

| Account | Balance (June 30, 2011) |
|---|---|
| Sewer Reserve Fund | $184,075 |
| Sewer System Development Fund | $62,765 |
| **Total:** | **$246,840** |

### 8.1.4. Proposed Rate Structure

The information presented in the preceding subsections has been used to develop a proposed rate structure for the City of Toledo based on the planned improvements. In order to proceed with the planned improvements, the City will need to secure funding. Some grant funding may be available to the City. However, loans will be required for a significant portion of the cost as well. The amount borrowed and the loan terms will have a direct effect on the resulting user rates.

Funding options are further discussed in Sections 8.2 and 8.3. For the purposes of this analysis, we will assume that the entire project is financed with a loan through the Rural Development Administration. The present interest rate on loans through Rural Development is 3.375% per year and loan terms can be up to 40 years. We have provided analyses based on 20, 25, 30 and 40 year terms for comparison.

Any grant funding awarded to the City should be considered when finalizing the rate structure. Also, the interest rates and terms of any loans actually taken out will play a part in the final rates users are required to pay.

As mentioned above, the final rate structure will depend greatly on the funding package secured by the City, interest rates, current construction costs, and other potential variables.

## *8.2. Evaluation of Local Funding Resources*

A number of local funding sources are available to the City for sharing the cost of the planned wastewater treatment plant and conveyance system improvements. The amount and type of local funding obligations for infrastructure improvements will depend in part on the amount of grant funding anticipated and the requirements of potential loan funding. Local revenue sources for capital expenditures include various types of bonds, capital construction funds, system development charges, system user fees, and ad valorem taxes. Local revenue sources for operating costs include system user fees and ad valorem taxes. Each of these financing mechanisms is briefly described below along with the appropriateness of each for the improvements recommended in this Plan.

### 8.2.1. **General Obligation Bonds**

General Obligation (GO) bonds have the full faith and resources of the City behind them including property taxes, rate income, and other revenues to ensure that obligations are met. As a result of this backing, GO bonds often have a lower interest rate and are generally considered to have lower risk and are a more attractive investment in the municipal bond market. For a community to undertake a project funded with a GO bond, they must pass a vote of the people in order to sell the bonds. In some cases, communities spend a great deal of time, money and effort only to have the electorate reject the project by denying the GO bond funding measure. As a result, many communities shy away from GO bond funding options.

### 8.2.2. **Revenue Bonds**

Revenue Bonds (RB) are retired through revenues obtained through user rates and charges. They do not have the full faith of the community behind them in that property taxes and other forms of revenue are not pledged to retire the debt. As such, they are considered as a higher risk and often have slightly higher interests rates associated with them. However, as property taxes are not obligated, a vote of the public is not required for selling revenue bonds to fund a project. This often makes revenue bonds a preferred choice for public improvements.

Bonds sales, regardless of type, have several requirements and processes that must be met for the bond sale to move forward. These requirements vary but generally include:

- Project documentation to prove feasibility of the project and the funding plan.
- Assistance from a bond counsel agent
- Retain a year of payments, in reserve, to provide a level of confidence that the City will not default on their debt payments.
- The bond process includes issuance costs that increase the overall cost of a project.
- Other requirements and steps to negotiate the process of obtaining funding.

### 8.2.3. **Improvement Bonds**

Improvement (Bancroft) bonds can be issued under an Oregon law called the Bancroft Act. These bonds are an intermediate form of financing that is less than full-fledged general obligation or revenue bonds. This type of bond is quite useful, especially for smaller issuers or for limited purposes.

An improvement bond is payable only from the receipts of special benefit assessments, not from generally tax revenues. Such bonds are issued only where certain properties are recipients of special benefits not accruing to other properties. For a specific improvement, all property within the improvement area is assessed on an equal basis, regardless of whether it is developed or undeveloped. The assessment becomes a direct lien against the property, and owners have the option of either paying the assessment in cash or applying for improvement bonds. If the improvement bond option is taken, the City sells Bancroft improvement bonds to finance the construction, and the assessment is paid over 20 years in 40 semiannual installments with interest. Cities and special districts are limited to improvement bonds not exceeding 3% of true cash value.

With improvement bond financing, an improvement district is formed, boundaries are established, and the benefiting properties and property owners are determined. The engineer usually determines an approximate assessment, either on a square foot or front-foot basis. Property owners are then given an opportunity to object to the project assessments. The assessments against the properties are usually not levied until the actual cost of the project is determined. Since this determination is normally not possible until the project is completed, funds are not available from assessments for the purpose of making monthly payments to the contractor. Therefore, some method of interim financing must be arranged or a pre-assessment program based on the estimated total costs must be adopted. Commonly, warrants are issued to cover debts, with the warrants to be paid when the project is complete.

The primary disadvantage to this source of revenue is that the property to be assessed must have a true cash value at least equal to 50% of the total assessments to be levied. As a result, owners of undeveloped properties usually require a substantial cash payment. In addition, the development of an assessment district is very cumbersome and expensive when facilities for an entire community are contemplated. In comparison, general obligation bonds can be issued in lieu of improvement bonds and are usually more favorable.

### 8.2.4. **System Development Charges**

System development charges (SDC's) are fees collected as previously undeveloped property is developed. The fees are used to finance the necessary capital improvements and municipal services required by the development. Such fees can only be used to recover the capital costs of infrastructure improvements. Operating, maintenance, and replacement costs cannot be financed through SDC's.

Two types of charges are permitted under the Oregon Systems Development Charges Act: improvement fees, and reimbursement fees. SDC's that are charged before a project is undertaken are considered improvement fees and are used to finance capital improvements to be constructed. After construction, SDC's are considered reimbursement fees and are collected to recapture the costs associated with capital improvements already constructed or under construction. A reimbursement fee represents a charge for utilizing excess capacity in an existing facility paid for by others. The revenue generated by this fee is typically used to pay back existing loans for improvements.

Under the Oregon SDC Act, methodologies for deriving improvement and reimbursement fees must be documented and available for review by the public. A capital improvement plan must also be prepared which lists the capital improvements that may be funded with improvement fee revenues. The estimated cost and timing of each improvement also must be included in the capital improvement plan. Thus, revenue from the collection of SDC's can only be used to finance specific items listed in a capital improvement plan. In addition, SDC's cannot be assessed on portions of the project paid for with grant funding.

### 8.2.5. **Ad Valorem Taxes**

Ad valorem property taxes are often used as a revenue source for utility improvements. Property taxes may be levied on real estate, personal property, or both. Historically, ad valorem taxes were the traditional means of obtaining revenue to support all local governmental functions.

A major advantage of these taxes is the simplicity of the system. It requires no monitoring program for developing charges, additional accounting and billing work is minimal, and default on payments is rare. In addition, ad valorem taxation provides a means of financing that reaches all property owners that benefit from a wastewater system, whether a property is developed or not. The construction costs for a project are shared proportionally among all property owners based on the assessed value of each property.

Depending on the project, ad valorem taxation may result in property owners paying a disproportionate share of the project costs compared to the benefits received. Public hearings and an election with voter approval would be required to implement ad valorem taxation.

#### 8.2.6. **System User Fees**

System user fees can be used to retire general obligation bonds and are commonly the sole source of revenue used to retire revenue bonds and to finance operation and maintenance of a system. System user fees represent charges of all residences, businesses and other users that are connected to the wastewater system. These fees are established by resolution and may be modified as needed to account for increased or decreased operating and maintenance costs. User fees may be based on a metered volume of water consumption and/or on the type of user (i.e. residential, commercial, industrial, etc.).

#### 8.2.7. **Assessments**

Under special circumstances, the beneficiary of a public works improvement may be assessed for the cost of a project. For example, the City may provide some improvements or services that directly benefit a particular development. The City may choose to assess the developer to provide up-front capital to pay for the improvements.

## *8.3. Evaluation of Federal and State Funding Resources*

Some level of outside funding assistance in the form of grants or low interest loans may be necessary to make the proposed improvement projects affordable for the City of Toledo and its citizens. The amount and types of outside funding will dictate the amount of local funding that the City must secure. In evaluating grant and local programs, the major objective is to select a program or combination of programs that is available and the most beneficial for the planned project.

This section provides a brief description of the major Federal and State funding programs that are typically utilized to assist qualifying communities in the financing of infrastructure improvement projects. Each of the government assistance programs has certain prerequisites and requirements in order for a community to qualify. The assistance programs promote goals such as aiding economic development, benefiting areas of low to moderate income families, and providing for specific community improvement projects. Because each program has specific requirements, not all communities or projects will qualify for each of the programs.

#### 8.3.1. **Economic Development Administration Public Works Grant Program**

The EDA Public Works Grant Program, administered by the U.S. Department of Commerce, is aimed at projects which directly create permanent jobs or remove impediments to job creation in the project area.

Thus, to be eligible for this grant, a community must be able to demonstrate the potential to create jobs from the project. Potential job creation is assessed with a survey of businesses to demonstrate the prospective number of jobs that might be created if the proposed project is completed.

Projects must be located within an EDA designated Economic Development District. Priority is given to projects that improve opportunities for the establishment or expansion of industry and which create or retain both short-term and long-term private sector jobs. Communities that can demonstrate that the existing system is at capacity (i.e. moratorium on new connections) have a greater chance of being awarded this type of grant. EDA grants are usually in the range of 50 to 80 percent of the project cost. Therefore, some type of local funding also is required. Grants typically do not exceed one million dollars.

### 8.3.2. **Water and Waste Disposal Loans and Grants (Rural Development)**

The Rural Utilities Service administers a water and wastewater loan and grant program designed to improve the quality of life and promote economic development in rural America. The Rural Utilities Service programs provide needed facilities to ensure health and safety and stimulate local economy by allowing access to new and advanced services and job opportunities. Program funds can be used for water, sewer, solid waste, and storm drainage projects. The most common uses are to restore deteriorating water supplies, or to improve, enlarge, or modify inadequate water or waste facilities.

Eligible applicants for Rural Utilities funds include public bodies and Indian Tribes. Non-profit corporations with significant ties to the local rural community may also be eligible. Funding is targeted to rural areas with populations of 10,000 or less. Applicants must be unable to obtain commercial financing at reasonable rates and terms or finance the project from existing resources.

The proposed project must serve a rural area not likely to decline in population below that for which the project is designed. The project should serve the present population and provide for foreseeable growth. Proposed projects should be necessary for orderly community development consistent with a comprehensive community or county development plan. Facilities must be modest in design, size, and cost. Water meters, a primary instrument for promoting conservation, are required by the agency. All water and wastewater systems must meet the standards set by the State Department of Environmental Quality.

The Rural Utilities staff review each project to determine need based on various priority points. Prioritization is necessary due to limited funding and to make sure the most deserving projects receive assistance.

When possible, loan funds are combined with other federal and state financing to reduce the end cost to users of the system. Depending on median household income (MHI) and need, communities may qualify for grant funds of up to 75% of the eligible project costs. These grants can help reduce water and waste disposal rates to reasonable levels. Rural Utilities loans have a term of up to 40 years or for the useful life of the facility, whichever is less.

There are three different interest rates available for Rural Utilities loans:

- ***Poverty Line Rate.*** The poverty line rate of 2.0% per annum applies to communities with a MHI below the state poverty level or 80% of the state non-metropolitan median household income (SNMHI). There must also be a health standard violation to receive the poverty loan rate (Rate is for quarter ending June 30, 2012).

- ***Intermediate Rate.*** The intermediate rate applies to projects in communities that are not eligible for the poverty rate and have a MHI between SNMHI and 80% of SNMHI. The intermediate interest rate is set halfway between the poverty line interest rate and the market rate.

- ***Market Rate.*** The market rate applies to projects in communities who do not qualify for the lower rates and who have MHI exceeding 100 % of the SNMHI for the state. The agency sets the intermediate and market rates quarterly, based on the bond market. The final rate for the project is the lowest rate in effect at the time of loan approval or closing.

To ensure the federal investment, the best security position practicable must be acquired. Acceptable forms of security for utility systems and public bodies include revenue bonds; other pledges of taxes or assessments; general obligation bonds; and assignment of income.

Grant fund eligibility is determined based on population, MHI, and user rates. Priority for grant funding is given to projects with populations of less than 5,500. Communities with low MHI may receive grant funding to reduce user costs to a reasonable level for rural residents. User rates are considered reasonable if they are less than or equal to existing prevailing rates in similar communities with similar systems.

Total grant funding cannot exceed the following percentages of eligible project development costs:

- 75% when the community meets poverty line interest rate criteria;
- 45% when the community meets intermediate interest rate criteria.

Maximum grant amounts based on MHI are provided in the following table.

**Table 8.3.2 – Maximum Rural Development Grant Funds based on MHI**

| Median Household Income (MHI) | Meets Criteria for Health or Sanitary Concern | Maximum Grant | Interest Rate [a] |
|---|---|---|---|
| <$40,447 | Yes | 75% | 2.0% (Poverty Rate) |
| <$40,447 | No | 45% | 2.75% (Intermediate Rate) |
| $40,447 - $50,559 | N/A | 45% | 2.75 % (Intermediate Rate) |
| >$50,559 | N/A | 0% | 3.375% (Market Rate) |

[a] Rates apply for quarter ending June 30, 2012.

The MHI of Lincoln County reported from 2007-2011 Census data was $41,764. At that time, the MHI statewide was $49,850. Based on the cited MHI Lincoln County which the City of Toledo is located in, it is estimated that the City would qualify for some grant assistance from Rural Development.

There are other restrictions and requirements associated with these loans and grants. If the City becomes eligible for grant assistance, the grant will apply only to eligible project costs. Additionally, grant funds are only available after the City has incurred long-term debt resulting in an annual debt service obligation equal to 0.5% of the MHI. In addition, an annual funding allocation limits the Rural Development funds. To receive a Rural Development loan, the City must secure bonding authority, usually in the form of general obligation bonds or revenue bonds.

### 8.3.3. **Oregon Community Development Block Grant Program**

Since the late 1980's the state of Oregon has administered the U.S. Department of Housing and Urban Development's Community Development Block Grant (CDBG) funds for the non-entitlement cities and counties of the state. The primary objective of the program is the development of viable (livable) urban communities by expanding economic opportunities and providing decent housing and a suitable living environment principally for persons of low- and moderate-income. Each year the state develops an annual "Method of Distribution" which establishes how the funds will be used for that calendar year. The Method of Distribution can be found on the department's web site.

Under the 2012 CDBG Method of Distribution improvements to public water and wastewater systems are eligible for funding. To receive a grant the applicant must meet the following minimum criteria:

- Must be a City or County located in a non-metropolitan area of Oregon.
- Have over 51% of the population considered low- to moderate-income in the target area based on census data or a local survey.
- Annual waste disposal rates must be equal to or greater than the cost to handle an average of 7,500 gallons per residential connection per month.
- Use the funds to benefit current residents

Grant funding is subject to the applicant need, availability of funds and any other restrictions in the 2012 Method of Distribution. Under the 2012 program, a maximum grant amount of $2,000,000 is available for water and wastewater improvement projects. Applications for the CDBG program are accepted on a year round basis and evaluated quarterly in a competitive review process.

Toledo has 41.0% of the population listed as low- or moderate-income based on the 2000 US Census and is not eligible for funding under this program. The City may wish to perform a local survey of residents within the area affected by the project if it is thought that the results would be more favorable than that of the Census.

For additional information on the CDBG program, call (503) 986-0123 or visit the OECDD website at http://www.econ.state.or.us/cdbg.htm.

### 8.3.4. Special Public Works Fund

The Special Public Works Fund program provides funding for the infrastructure that supports job creation in Oregon. Loans and grants are made to eligible public entities for the purpose of studying, designing and building public infrastructure that leads to job creation or retention.

The public entities or "municipalities" that are eligible to apply for Special Public Works Fund assistance include:

- Cities
- Counties
- Ports incorporated under ORS 777.005 to 777.725 and 777.915 to 777.953 and under 778.010
- Domestic water supply districts organized under ORS chapter 264
- Sanitary districts organized under ORS 450.005 to 450.245

- Sanitary authority, water authority or joint water and sanitary authority organized under ORS 450.600 to 450.989
- County service districts organized under ORS chapter 451
- Tribal Councils of Indian Tribes in Oregon
- Airport district organized under ORS Chapter 838
- A district as defined in ORS 198.010 (see Appendix B for the specific list)

In order to be eligible, the proposed project must be owned by a public entity that is an eligible applicant. Examples of the many types of eligible municipally owned projects are listed below, although this is not a comprehensive list.

- Airport facilities
- Telecommunications infrastructure
- Port facilities, wharves and docks
- Railroads
- Buildings and associated equipment
- Solid waste disposal sites
- Acquisition of land
- Mitigation of environmental conditions
- Purchase of rights of way and easements necessary for infrastructure
- Roadways, bridges, etc.
- Storm drainage systems
- Wastewater systems
- Water systems
- The acquisition or construction of related equipment and fixtures

The Special Public Works Fund is comprehensive in terms of the types of project costs that can be financed. As well as actual construction, eligible project costs can include costs incurred in conducting feasibility and other preliminary studies and for the design and construction engineering.

The Fund is primarily a loan program. Grants can be awarded, up to the program limits, based on job creation or on a financial analysis of the applicant's capacity for carrying debt financing.

The total loan amount per project cannot exceed $10 million. The department is able to offer very attractive interest rates that typically reflect low market rates. In addition, the department absorbs the associated costs of debt issuance thereby saving applicants even more on the overall cost of borrowing. Loans are generally limited to the usable life of the contracted project, or 25 years from the year of project completion, whichever is less.

For infrastructure projects, grants are offered to projects creating or retaining jobs and are eligible for up to $5,000 per job created or retained. If a grant is offered it cannot exceed 85 percent of the project cost or $500,000, whichever is less. Additional grants may be awarded if there is a gap between the grant for jobs plus the loan and the total project costs.

For more information on the Special Public Works Fund program, call (503) 986-0123 or visit the OECDD website at http://www.econ.state.or.us/spwf.htm.

### 8.3.5. **Water/Wastewater Financing Program**

The Water/Wastewater Fund was created by the Oregon State Legislature in 1993. It was initially capitalized with lottery funds appropriated each biennium and with the sale of state revenue bonds since 1999. The purpose of the program is to provide financing for the design and construction of public infrastructure needed to ensure compliance with the Safe Drinking Water Act or the Clean Water Act.

The public entities that are eligible to apply for the program include: Cities, Counties, County Service districts (organized under ORS Chapter 451), Tribal Councils of Indian tribes, Ports, and Special Districts as defined in ORS 198.010.

Eligible activities include reasonable costs for construction improvement or expansion of drinking water, wastewater or storm water systems. Eligible projects include those related to drinking water source, treatment, storage and distribution; wastewater collection and capacity; stormwater system; purchase of rights-of-way and easements necessary for construction; and design and construction engineering. All projects must ensure that municipal water and wastewater systems comply with the Safe Drinking Water Act or the Clean Water Act.

To be eligible a system must have received, or is likely to soon receive, a Notice of Non-Compliance by the appropriate regulatory agency, associated with the Safe Drinking Water Act or the Clean Water Act. Projects also must meet other state or federal water quality statutes and standards.

Ineligible projects include privately owned facilities and infrastructure; purchase of property not related to infrastructure construction; costs incurred prior to award, except costs for engineering and other support activities necessary to construction.

The Fund provides both loans and grants, but it is primarily a loan program. The loan/grant amounts are determined by a financial analysis of the applicant's ability to afford a loan (debt capacity, repayment sources and other factors).

The Water/Wastewater Financing Program's guidelines, project administration, loan terms and interest rates are similar to the Special Public Works Fund program. The maximum loan term is 25 years or the useful life of the infrastructure financed, whichever is less. The maximum loan amount is $10,000,000 per project through a combination of direct and/or bond funded loans. Loans are generally repaid with utility revenues or voter approved bond issues. A limited tax obligation pledge may also be required. "Credit worthy" borrowers may be funded through sale of state revenue bonds.

Grant awards can be awarded up to a maximum of $750,000 depending on a financial review. An applicant is not eligible for grant funds if the annual median household income in the affected area is equal or greater than 100 percent of the state average median household income for the same year.

Technical assistance funding for preliminary planning, engineering studies and economic investigations are available to municipalities with populations under 15,000 residents. Technical assistance projects must be done in preparation for an eligible construction project and can be awarded loans of up to $50,000 or grants of up to $20,000 per project.

For more information on the Special Public Works Fund program, call (503) 986-0123 or visit the OECDD website at http://www.econ.state.or.us/wtrww.htm.

### 8.3.6. **Clean Water State Revolving Fund (CWSRF)**

The Clean Water State Revolving Fund (CWSRF) Loan Program administered by the Oregon Department of Environmental Quality (DEQ) provides low-cost loans for the planning, design and construction of a variety of projects that address water pollution. The loans through the CWSRF program are available to Oregon's public agencies, including cities, counties, sanitary districts, soil and water conservation districts, irrigation districts and various special districts.

Congress established the CWSRF in 1987, to replace the Construction Grants program, which had provided direct grants to communities to complete sewer infrastructure projects. The CWSRF program provides several types of loans and varying interest rates. Currently, loans are available with terms of 5 years at 0.97% APR to 20 years at 2.52% APR.

There are six different types of loans available within the program. These include traditional planning, design and construction loans. There are also loans available for emergencies, urgent repairs and local community projects. Each of these loan types has different financial terms, and is intended to provide communities with choices when financing water quality improvements. Interest rates are based on the nation's bond buyer's index and fluctuate quarterly. The interest rates of various loans are substantially discounted from the bond rate. For example, with a quarterly bond rate of 5.0%, the CWSRF interest rates (depending on the type of loan) would range from 0.97% to 3.88%. Loan payback periods vary, ranging from 5 to 20 years. Loans do include an annual loan fee of 0.5% of the outstanding balance. Planning loans are exempt from this fee.

Eligible projects include:

- Wastewater system plans and studies
- Secondary or advanced wastewater treatment facilities
- Irrigation improvements
- Infiltration and inflow correction
- Major sewer replacement and rehabilitation
- Qualified storm water control
- Onsite wastewater system repairs
- Matching funds for some U.S. Department of Agriculture conservation programs
- Estuary management efforts
- Various nonpoint source projects (stream restorations, animal waste management, conservation easements)
- Qualified brownfields projects

All eligible proposed projects are ranked based upon their application information and entered on the program's Project Priority List. Points are assigned based on specific ranking criteria. Newly ranked projects are integrated into the priority list on a regular basis. The Project Priority List is incorporated within DEQ's annual Intended Use Plan which indicates the proposed use of the funds each year.

Projects are funded based on the availability of loan monies. If monies are insufficient to fund all the approved projects, funds are distributed to as many projects as possible based on the Project Priority List. Each time new monies become available, those monies are allocated to as many unfunded or partially funded projects as possible.

For additional information on the CWSRF loan program, call (800) 452-4011 or visit the DEQ website at http://www.deq.state.or.us/wq/loans/loans.htm.

### 8.3.7. **Oregon Department of Energy, Small Scale Energy Loan Program (SELP)**

The purpose of the Energy Loan Program (also known as SELP) is to promote energy conservation and renewable energy resource development. The Energy Loan Program can loan to individuals, businesses, schools, cities, counties, special districts, state and federal agencies, public corporations, cooperatives, tribes, and non-profits in Oregon.

The program offers low-interest loans for projects that:

- Save energy
- Produce energy from renewable resources such as water, wind, geothermal, solar, biomass, waste materials or waste heat
- Use recycled materials to create products
- Use alternative fuels

Current loan rates for cities vary depending on the bond market, term of loan. Loans also include an application fee of 0.1%, an underwriting fee of 0.5%, and a loan fee of 1.0% of the loan amount.

For more information on the SELP program, call (503) 503-2123 or visit the Oregon Department of Energy website at http://www.oregon.gov/ENERGY/LOANS/index.shtml.

## *8.4. Recommended Rate Structure and Financing Strategy*

A financing strategy or plan must provide a mechanism to generate capital funds in sufficient amounts to pay for the proposed improvements over the relatively short duration in design and construction, generally two years. The financing strategy must also identify the manner in which annual revenue will be generated to cover the expense for long-term debt repayment and the on-going operation and maintenance of the system.

The objectives of a financial strategy include the following:

- Identify the capital improvement cost for the project and the estimated expenses for operation and maintenance.

- Evaluate potential funding sources and select the most favorable program.

- Identify the local cost share based on the amount of outside funding obtained.

- Determine the cost to system users to finance the local share and the annual cost for operation and maintenance.

### 8.4.1. **Funding Sources**

With any of the funding sources listed within Sections 8.2 and 8.3 the City is advised to confirm specific funding amounts with the appropriate agencies prior to making local financing arrangements. A one-stop meeting with funding agencies is recommended as soon as the City has made a firm commitment as to the schedule and extent of capital improvements.

Most of the grant programs require that the project address a DEQ issued violation or order before the project is eligible for funding. Rural Development will issue grants for projects without this requirement, but for a reduced amount and the project must pass strict scrutiny.

# 9.0 Recommended Plan

Section

9

This Section is intended to summarize all of the recommendations in this Facilities Plan and provide clear and concise information on project selection, capacity needs, project prioritization, design parameters, project costs, and financing strategies.

This Section shall outline the recommended plan for both the collection system and the wastewater treatment system.

## *9.1. Introduction*

Through the analyses and studies that were completed within this facilities planning effort, numerous project recommendations have been developed. These recommendations include improvements to the wastewater treatment facilities in Toledo as well as improvements to the City's wastewater collection system.

As the projects vary in their criticality, the projects have been divided into three separate and distinct priority groups. The priority groups are further described below:

Priority 1 Projects: Priority 1 projects are the most critical and should be undertaken as soon as possible in order to meet DEQ requirements. Priority 1 projects should be considered as the most immediate needs for the City's wastewater system.

Priority 2 Projects: Projects that should be undertaken within the first half of the planning period to restore aging facilities to newer operating conditions. While they do not have to be undertaken immediately, the City should include them in their capital improvement plans and obtain funding to undertake these projects.

Priority 3 Projects: Priority 3 projects are projects that are primarily dependent on development and expansion of the collection system to provide sewer service to new areas. Priority 3 projects are most likely to be driven by development and the need to expand the collection system to service new properties and new subdivisions. Funding for Priority 3 projects are likely to be financed through a combination of City funds, SDC funds, and developer contributions. As these projects are likely to be development driven, they need not be scheduled for implementation. They should, however, be included within the CIP and considered within any wastewater SDC methodology developed by the City.

With these priorities in mind, the remainder of this section will further describe the recommended projects, their costs and design criteria, and financing strategies for the recommended projects.

### 9.1.1. **Project Selection**

Within this section, project selection descriptions will be provided for each priority group. Additional information on each recommended project is available within Section 7 of this facilities plan.

**Priority 1 Projects:**

The following projects are selected as priority 1 projects:

- **Wastewater Treatment Facility Improvements:** It is recommended that the City construct improvements to remedy the wastewater treatment facility deficiencies. The upgrades to the treatment facility should include a number of improvement components to improve operations of the facility. The treatment facility improvements should include the following major components:

    - **Headworks:** Redesign and replace the removed flow equalization weir.

    - **Effluent Booster Pumps:** Install high capacity, low head propeller pumps to increase discharge during high tide events.

    - **Outfall:** Replace northernmost 300' of outfall pipe.

    - **Sludge Handling and Storage:** As part of the City's sludge disposal plan, the new facility should include a new sludge storage tank.

- **Lift Station Improvements:** The next Priority 1 improvement projects involve completing improvements necessary at the City's Wastewater Lift Station. The following series of improvement projects are at the following lift stations:

    - **Butler Bridge Station Improvements:** Improvements to the station itself include the installation of a new wetwell adjacent to the existing station so that new submersible pumps can be utilized and the old wet well/dry well system can be eliminated along with the confined space entry and other operational issues. It is recommended that a new building be constructed to house the electrical and control equipment and that the existing generator be re-installed at the site to meet DEQ reliability requirements.

    - **Butler Bridge Lift Station Force Main:** As part of the Butler Bridge Lift Station upgrades, it is also recommended that the old portion (~1100 ft) of the existing force main be replaced with a new 14-inch force main.

    - **Ammon Road Lift Station Improvements:** Improvements to the station itself include the installation of a new wetwell adjacent to the existing station so that new submersible pumps can be utilized and the old wet well/dry well system can be eliminated along with the confined space entry and other operational issues. It is recommended that a new building be constructed to house the electrical and control equipment and that the existing generator be re-installed at the site to meet DEQ reliability requirements.

- **Gravity Collection System Improvements:** The final Priority 1 improvement projects identified involve completing necessary improvements to the City's gravity wastewater collection system. These improvements where identified and prioritized in the I&I investigation report which is provided in Appendix C. Below is a general description of the type of improvements required:

    - **Pipe Improvements:** Improvements to the gravity systems existing collection pipes include: pipe replacement, lining, pipe bursting, and pipeline patches. For a more detailed breakdown of the proposed improvements and their locations within the collection system please refer to the I&I study provided in Appendix C.

- **Manhole Improvements:** Improvements to the gravity systems existing manholes include: replacement, lining, patches, and grouting of the systems manholes. For a more detailed breakdown of the proposed improvements and their locations within the collection system please refer to the I&I study provided in Appendix C.

**Priority 2 Projects:**

The following projects have been grouped together as Priority 2 projects:

- **Lift Station Improvements:** The following series of improvement projects have been identified as Priority 2 projects and are located at the following lift stations:
  - **"A" Street Lift Station Improvements:** Basic improvements are recommended for the "A" Street Lift Station including upgrading piping, pumps, fittings, structural upgrades, electrical and control systems. The upgrades are intended to extend the life of the facility and improve the operation and maintenance issues related to the pump station.
  - **"A" Street Lift Station Force Main:** As part of the "A" Street Lift Station upgrades, it is also recommended that the facilities existing force main be replaced with a new 12-inch force main.
- **Gravity Collection System Improvements:** The final Priority 2 improvement projects identified involve completing necessary improvements to the City's gravity wastewater collection system. These improvements where identified and prioritized in the I&I investigation report which is provided in Appendix C.

**Priority 3 Projects:**

The following projects have been grouped together as Priority 3 projects:

- **Lift Station Improvements:** The following series of improvement projects have been identified as Priority 2 projects and are located at the following lift stations:
  - **High School Lift Station Improvements:** Basic upgrades are recommended for the High School Lift Station. Improvement recommendations include piping and fitting upgrades, generator installation, controls and electronic upgrades and structural upgrades. These recommendations are intended to extend the useful life of the pump station through and beyond the planning period.
  - **Lincoln Way Lift Station Improvements:** Basic upgrades are recommended for the Lincoln Way Lift Station. Improvement recommendations include piping and fitting upgrades, generator installation, controls and electronic upgrades and structural upgrades. These recommendations are intended to extend the useful life of the pump station through and beyond the planning period.
- **Gravity Collection System Improvements:** The final Priority 3 improvement projects identified involve completing necessary improvements to the City's gravity wastewater

collection system. These improvements where identified and prioritized in the I&I investigation report as both priority level 3 and 4, a copy of the I&I is provided in Appendix C, but are combined into a single priority level for inclusion into this report.

### 9.1.2. **Project Cost Summary**

Three project priority groups have been developed in Section 9. As mentioned previously, the projects vary in their criticality with some requiring that they be undertaken as soon as possible while others can be planned for and undertaken later in the planning period.

A summary of the recommended projects costs is provided in the table below for all three project priority categories. Detailed cost estimates are provided in Section 7.

**Table 9.1.3 - Recommended Project Cost Summary**

| Recommended Improvements and Alternatives: | | | |
|---|---|---|---|
| **Priority 1 Projects:** | | | |
| **Facility** | **Alternative, Recommendation** | **Description** | **Total Cost** |
| Wastewater Treatment Plant | Headworks | New Flow Equalization Weir | $25,000 |
| | Outfall Pipe | Replace Portion of Outfall | $207,230 |
| | Effluent Booster Pumps | Install Effluent Booster pumps | $246,935 |
| | Sludge Alternative A | Sludge Storage Tank | $514,829 |
| Ammon Road Lift Station | Alternative B | New Wet Well | $1,303,543 |
| Butler Bridge Lift Station | Alternative B | New Wet Well | $1,404,767 |
| Butler Bridge Force Main | Recommendation | Replace Portion of Force Main | $262,049 |
| Collection System (Piping and Manholes) | I & I - Priority 1 | Pipe Replacement, Lining, Bursting or Patching; Manhole Rehabilitation | $380,935 |
| | | Total Priority 1 Projects: | $4,345,288 |
| **Priority 2 Projects:** | | | |
| **Facility** | **Alternative, Recommendation** | **Description** | **Total Cost** |
| "A" Street Lift Station | Alternative A | Dry Pit Upgrade | $671,248 |
| "A" Street Lift Station Force Main | Recommendation | Replace Force Main | $172,175 |
| Collection System (Piping and Manholes) | I & I - Priority 2 | Pipe Replacement, Lining, Bursting or Patching; Manhole Rehabilitation | $565,400 |
| | | Total Priority 2 Projects: | $1,408,823 |
| **Priority 3 Projects:** | | | |
| **Facility** | **Alternative, Recommendation** | **Description** | **Total Cost** |
| High School Lift Station | Alternative B | Upgrades and Life Extension Improvements | $233,651 |
| Hospital Lift Station | Alternative B | Upgrades and Life Extension Improvements | $148,928 |
| Collection System (Piping and Manholes) | I & I - Priority 3 & 4 | Pipe Replacement, Lining, Bursting or Patching; Manhole Rehabilitation | $490,340 |
| | | Total Priority 3 Projects: | $872,919 |
| | | **Total Overall Plan Cost:** | **$6,627,030** |

## *9.2. Financing Strategy*

The City of Toledo must upgrade and improve their wastewater facilities in order to provide reliable wastewater conveyance and treatment for their system for upcoming planning period and beyond.

This wastewater facilities plan outlines a plan for all necessary improvements and represents a significant investment for the City in new wastewater treatment facilities and conveyance system improvements. The City must develop a strategy and plan for financing the recommended improvements.

Section 8 of this facilities plan outlines a number of financing options that are available to the City for financing the recommended improvements. The financing options include local funding sources, state and federal loan and grant programs, tax programs, and others. While the ultimate financing package that the City will ultimately utilize depends on the results of coordination with the various funding agencies, this section will summarize the general direction the City should proceed with and provide some insight into the potential impacts to rate payers.

### 9.2.1. Project Expenses

As outlined earlier in this Section, improvement projects recommended in this facilities plan total more than six million dollars. The projects have been grouped into three main priority categories with only the Priority 1 projects being identified as having the most critical and immediate need.

Of the total project costs recommended, the Priority 1 projects total approximately four million dollars and include all of the recommended wastewater treatment plant improvements, necessary upgrades to gravity collection system and the Ammon Road and Butler Bridge Lift Station Facilities.

### 9.2.2. Financing Strategy

The City should proceed with the following steps as they move forward with the financing strategy for the wastewater improvement projects:

1. As soon as the City receives approval for the completed Toledo Wastewater Facilities Plan, the City should contact OECDD and DEQ to schedule a one-stop meeting. At this one-stop meeting, all of the potential agencies who may be able to provide funding will send representatives to discuss the City's funding needs and develop a funding package for the improvement projects. The agencies will, in real time, make recommendations and will discuss what each agency can offer. The result will be a funding package made up of grants and loans from a number of agencies to fund the project.

2. Following the one-stop meeting, the City should immediately process the necessary paperwork to apply for the funding included in the funding package recommended at the one-stop meeting. This will require numerous applications and other administrative efforts to apply for funding. The City should apply to any and all programs or agencies that have the potential to provide grant money to reduce the impact to rate payers.

3. Due to the magnitude of the required improvements, the City will not likely receive grants sufficient to cover all of the costs of the projects. In fact, the City will most likely be required to take out loans for a significant portion of the project costs. These loans will be paid back over a period of time that can likely be extended to as much as 40 years, though the final loan period will depend on the funding agency and their policies on payback. Because the City will have to pay back loan monies, a rate increase will be required to generate the revenue to pay back the loans. The City should immediately set up a timeline and plan for rate increases. The plan should include efforts to educate the public and provide for public meetings and other opportunities for the public to learn about the upcoming improvement projects, the project need, and the project costs.

4. Once the City receives notification that they have secured the necessary funding to complete the work, they can complete design activities in preparation for bidding and construction of the improvements.

### 9.2.3. **Impact to Rate Payers**

As mentioned above, the funding package for the recommended project will include a loan component that will necessitate a rate increase for the average rate payer. While the final funding package will not be known until after the one-stop meeting and not confirmed until the City receives notice that they have secured the necessary funding, it is important that the City be provided with some insight on the potential impact to rate payers so that they may begin educating the public and develop plans for increasing rates as needed to pay for the significant costs associated with these improvements.

To complete the Priority 1 Improvements, a loan is assumed with a 20-year payback at 3.00%. Select agencies may offer lower rates and/or longer a repayment period, but for this exercise the above assumption is made. Any lower rates or longer repayment period would lessen the required rate increases. Given the terms identified above, an additional $26,800 per month will be needed to repay the loan (with 10% additional fund cushion). According to the discussion on Section 3.4 there are 1531 Equivalent Dwelling Units (EDU's) in the City which means that there needs to be an increase of approximately **$17.49 per EDU**. This can be either be added to the base rate (currently $11.20 per month) or as in increase to the 'per one thousand gallons of treated water' usage rate (currently $14.83 per 1000 gal).

To complete the Priority 2 Improvements, using the same loan assumptions as phase 1, but with expected project cost increases due to inflation (based on ENR Construction Cost Index) at a recent average rate of 3% per year, the required rate increase is an <u>additional</u> **$6.34 per EDU**.

To complete the Priority 3 Improvements, the required rate increase is an <u>additional</u> **$4.98 per EDU**.

Having explored the potential worst case scenario for the impact to rate payers, most likely the City will qualify for and receive some grant monies for the project. It must be understood that grant monies have become increasingly difficult to obtain and the total awards to communities have decreased over the years.

As mentioned before, the final impact to rate payers will not be known until the final funding package is confirmed and all variables are set. Should interest rates rise significantly before the funding package is secured, the impact to rate payers will be greater.

The City should begin in earnest in educating the public, developing a rate increase plan, and pursuing grant and loan monies.

## *9.3. Implementation Schedule*

Implementation for the recommended projects in this plan relies on obtaining funds and following a schedule that is, for the most part, governed by the City's schedule.

The City has already begun the process of implementation of a plan to upgrade their system by completing this wastewater facilities plan. The City must continue to take the steps necessary and stay on schedule to implement the recommended improvements contained within the plan.

The following milestones and activities should be considered as steps on the path of implementation:

| Milestone or Implementation Step | Date |
|---|---|
| 1. Complete facilities planning | Winter, 2014 |
| 2. Begin funding acquisition process | Spring 2014 |
| 3. DEQ Review complete and approval of Facilities Plan (estimated) | Spring 2014 |
| 4. Schedule One-Stop Meeting | Spring 2014 |
| 5. Complete funding applications | Summer 2014 |
| 6. Obtain final funding package | Fall 2014 |
| 7. Begin predesign activities for Priority 1 projects | Spring 2014 |
| 8. Submit predesign report to DEQ for approval | Summer 2014 |
| 9. Begin design phase of Priority 1 projects | Summer 2014 |
| 10. Complete design of Priority 1 projects and submit for DEQ approval | Winter 2014 |
| 11. Address DEQ comments and complete final construction documents | Spring 2015 |
| 12. Advertise for bids for construction of Priority 1 projects | Spring 2015 |
| 13. Begin construction of Priority 1 projects | Summer 2015 |

# APPENDIX A

APPENDIX A

ISSUED



**NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM**
**WASTE DISCHARGE PERMIT**

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

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

**ISSUED TO:**
City of Toledo
PO Box 220
Toledo, OR 97391

**SOURCES COVERED BY THIS PERMIT:**

| Type of Waste | Outfall Number | Outfall Location |
|---|---|---|
| Treated Wastewater | 001 | R.M. 13.7 |

**FACILITY TYPE AND LOCATION:**
Activated Sludge
City of Toledo
1105 SE Fir Street, Toledo
**Treatment System Class:** Level III
**Collection System Class:** Level II

**RECEIVING STREAM INFORMATION:**
Basin: Mid Coast
Sub-Basin: Siletz-Yaquina
Receiving Stream: Yaquina River
LLID: 1240830446097 13.7 D
County: Lincoln

**EPA REFERENCE NO:** OR-002086-9

Issued in response to Application No. 982958 received August 23, 2004. This permit is issued based on the land use findings in the permit record.

Timothy C. McFetridge, (Acting) Western Region Water Quality Manager

December 27, 2005
Date

## PERMITTED ACTIVITIES

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



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

**SCHEDULE A**

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

a. Treated Effluent Outfall 001

(1) May 1 - October 31:

| Parameter | Average Effluent Concentrations Monthly | Weekly | Monthly* Average lb/day | Weekly* Average lb/day | Daily* Maximum lbs |
|---|---|---|---|---|---|
| $BOD_5$ | 10 mg/L | 15 mg/L | 61 | 91 | 120 |
| TSS | 10 mg/L | 15 mg/L | 61 | 91 | 120 |

(2) November 1 - April 30:

| Parameter | Average Effluent Concentrations Monthly | Weekly | Monthly* Average lb/day | Weekly* Average lb/day | Daily* Maximum lbs |
|---|---|---|---|---|---|
| $BOD_5$ | 20 mg/L | 30 mg/L | 270 | 410 | 550 |
| TSS | 20 mg/L | 30 mg/L | 270 | 410 | 550 |

* Average dry weather design flow to the facility equals 0.73 MGD. Summer mass load limits based upon average dry weather design flow to the facility. Winter mass load limits based upon average wet weather design flow to the facility equaling 1.64 MGD. The daily mass load limit is suspended on any day in which the flow to the treatment facility exceeds 1.46 MGD (twice the design average dry weather flow).

(3)

| Other parameters (year-round) | Limitations |
|---|---|
| Fecal Coliform Bacteria | Shall not exceed a 30 day log mean of 100 organisms per 100 mL and a weekly log mean of 200 organisms per 100 mL. **(See Note 1)** |
| pH | Shall be within the range of 6.0 - 9.0 |
| $BOD_5$ and TSS Removal Efficiency | Shall not be less than 85% monthly average |
| Total Chlorine Residual | Shall not exceed 0.01 mg/L monthly average and 0.02 mg/L daily maximum **(See Notes 2 and 3)** |
| Excess Thermal Load (ETL) | Shall not exceed a weekly average of 11 million Kcals/day **(See Note 4)** |

(4) Except as provided for in OAR 340-045-0080, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-041-0245 except in the following defined mixing zone:

The allowable mixing zone is that portion of the Yaquina River extending out one hundred (100) feet from the east bank of the river and extending from a point one hundred (100) feet upstream of the outfall to a point one hundred (100) feet downstream from the outfall. The Zone of Immediate Dilution (ZID) shall be defined as that portion of the allowable mixing zone that is within ten (10) feet of the point of discharge.

b.

No wastes shall be discharged from these outfalls except as allowed in Schedule F, Section B, Condition 6 of this permit. If an overflow occurs between May 22 and June 1, and if the permittee demonstrates to the Department's satisfaction that no increase in risk to beneficial uses occurred because of the overflow, no violation shall be triggered if the storm associated with the overflow was greater than the one-in-five-year, 24-hour duration storm.

c. No activities shall be conducted that could cause an adverse impact on existing or potential beneficial uses of groundwater. All wastewater and process related residuals shall be managed and disposed in a manner that will prevent a violation of the Groundwater Quality Protection Rules (OAR 340-040).

**NOTES:**

1. At the point of discharge, the Yaquina River is water quality limited for bacteria year-round. A Total Maximum Daily Load (TMDL) has not been issued for these parameters at the time of permit issuance. Upon EPA approval of a TMDL addressing this pollutant, this permit may be reopened to include any Waste Load Allocation (WLA), best management practice or any other condition required by the TMDL.

2. When the total residual chlorine limitation is lower than 0.10 mg/L, the Department will use 0.10 mg/L as the compliance evaluation level (i.e. daily maximum concentrations below 0.10 mg/L will be considered in compliance with the limitations).

3. The total chlorine residual limitations shall not apply until completion of the compliance schedule in Schedule C Condition 3, or no later than the expiration date of this permit, whichever is sooner.

4. The thermal load limit was calculated using the average dry weather design flow and an estimated maximum weekly effluent temperature. The Excess Thermal Load limit is considered interim and may be adjusted up or down or eliminated when more accurate effluent temperature data becomes available. In addition, upon approval of a Total Maximum Daily Load for temperature for this sub-basin, this permit may be re-opened to include new or revised limits or other conditions or requirements regarding temperature and/or thermal loads.

## SCHEDULE B

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

a. Influent

The facility influent grab and composite samples and measurements are taken just after flow measurement prior to screening and grit removal.

| Item or Parameter | Minimum Frequency | Type of Sample |
|---|---|---|
| $BOD_5$ | 2/Week | Composite |
| TSS | 2/Week | Composite |
| pH | 3/Week | Grab |

b. Treated Effluent Outfall 001

The facility effluent grab and composite samples and measurements are taken from effluent box prior to discharge to Outfall 001.

| Item or Parameter | Minimum Frequency | Type of Sample |
|---|---|---|
| Total Flow (MGD) | Daily | Measurement |
| Flow Meter Calibration | Semi-Annual | Verification |
| $BOD_5$ and TSS | 2/Week | Composite |
| Pounds Discharged ($BOD_5$ and TSS) | 2/Week | Calculation |
| Fecal Coliform | Weekly | Grab |
| pH | 3/Week | Grab |
| Quantity Chlorine Used | Daily | Measurement |
| Total Chlorine Residual | Daily | Grab |
| Average Percent Removed ($BOD_5$ and TSS) | Monthly | Calculation |
| Test High Water Alarms | Twice per month | Other |
| Inspect Tide Gates | Weekly | Other |
| Effluent Temperature, Daily Maximum | Daily | Grab between 2-4 p.m. |
| Excess Thermal Load, seven day average | Weekly | Calculation **(see Note 1)** |
| Effluent Temperature, Average of Daily Maximums | Weekly | Calculation |

c. Biosolids Management

| Item or Parameter | Minimum Frequency | Type of Sample |
| --- | --- | --- |
| Sludge analysis including:<br>Total Solids (% dry wt.)<br>Volatile solids (% dry wt.)<br>Biosolids nitrogen for:<br>$NH_3$-N; $NO_3$-N; & TKN (% dry wt.)<br>Phosphorus (% dry wt.)<br>Potassium (% dry wt.)<br>pH (standard units)<br>Sludge metals content for:<br>As, Cd, Cu, Hg, Mo, Ni, Pb, Se & Zn, measured as total in mg/kg | Annually | Composite sample to be representative of the product to be land applied from the Sludge storage **(See Note 2)** |
| Record of locations where biosolids are applied on each DEQ approved site. (Site location maps to be maintained at treatment facility for review upon request by DEQ) | Each Occurrence | Date, volume & locations where sludges were applied recorded on site location map. |
| Record of % volatile solids reduction accomplished through stabilization | Monthly | Calculation **(See Note 3)** |
| Record of digestion days (mean cell residence time) | Monthly | Calculation **(See Note4)** |

## 2. Reporting Procedures

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

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

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

## 3. Report Submittals

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

b. For any year in which biosolids are land applied, a report shall be submitted to the Department by February 19 of the following year that describes solids handling activities for the previous year and includes, but is not limited to, the required information outlined in OAR 340-050-0035(6)(a)-(e).

**NOTES:**

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

   *(Weekly average of daily maximum effluent temperatures in ℃ - applicable stream temperature standard in ℃) X (Weekly average of daily flow in MGD) X 3.785 = Excess Thermal Load, in Million Kcals/day.*

2. Composite samples from the Sludge storage shall be taken from reference areas in the Sludge storage pursuant to Test Methods for Evaluating Solid Waste, Volume 2; Field Manual, Physical/Chemical Methods, November 1986, Third Edition, Chapter 9.

   Inorganic pollutant monitoring must be conducted according to Test Methods for Evaluating Solid Waste, Physical/Chemical Methods, Second Edition (1982) with Updates I and II and third Edition (1986) with Revision I.

3. Calculation of the % volatile solids reduction is to be based on comparison of a representative grab sample of total and volatile solids entering each digester (a weighted blend of the primary and secondary clarifier solids) and a representative composite sample of solids exiting each digester withdrawal line (as defined in note 1 above).

4. The days of digestion shall be calculated by dividing the effective digester volume by the average daily volume of sludge production.

## SCHEDULE C

Compliance Schedules and Conditions

1. By June 24, 2006 the permittee shall submit to the Department for review and approval an updated program and time schedule for identifying and reducing inflow. Within 60 days of receiving written Department comments, the permittee shall submit a final approvable program and time schedule. The program shall consist of the following:

   a. Identification of all overflow points and verification that sewer system overflows are not occurring up to a 24-hour, 5-year storm event or equivalent;

   b. Monitoring of all pump station overflow points;

   c. A program for identifying and removing all inflow sources into the permittee's sewer system over which the permittee has legal control; and

   d. If the permittee does not have the necessary legal authority for all portions of the sewer system or treatment facility, a program and schedule for gaining legal authority to require inflow reduction and a program and schedule for removing inflow sources.

2. The permittee shall complete the following schedule to comply with the Total Chlorine Residual limitations contained in Schedule A.1.a.(3):

   a. By no later than October 31, 2006 the permittee shall submit to the Department an evaluation of alternatives for corrective action that will result in compliance with the Total Chlorine Residual limit.

   b. By no later than October 31, 2008, the permittee shall submit to the Department for approval final engineering plans and specifications for the corrective actions necessary to comply with the Total Chlorine Residual limit.

   c. By no later than March 31, 2009, the permittee shall complete construction of all necessary improvements and comply with the Total Chlorine Residual limit.

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

## SCHEDULE D

**Special Conditions**

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

   All new biosolids application sites shall meet the site selection criteria set forth in OAR 340-050-0070 and must be located within Lincoln County. All currently approved sites are located in Lincoln County. No new public notice is required for the continued use of these currently approved sites. Property owners adjacent to any newly approved application sites shall be notified, in writing or by any method approved by DEQ, of the proposed activity prior to the start of application. For proposed new application sites that are deemed by the DEQ to be sensitive with respect to residential housing, runoff potential or threat to groundwater, an opportunity for public comment shall be provided in accordance with OAR 340-050-0030.

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

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

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

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

   b. The permittee's wastewater system may not be without supervision (as required by Special Condition 3.a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified at no less than one grade lower then the system classification.

   c. If the wastewater system has more than one daily shift, the permittee shall have the shift supervisor, if any, certified at no less than one grade lower than the system classification.

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

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

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

4. The permittee shall notify the DEQ Western Region – Coos Bay Office (phone: (541) 269-2721) in accordance with the response times noted in the General Conditions of this permit, of any malfunction so that corrective action can be coordinated between the permittee and the Department.

**SCHEDULE F**
**NPDES GENERAL CONDITION – DOMESTIC FACILITIES**

## SECTION A. STANDARD CONDITIONS

1. Duty to Comply with Permit
The permittee must comply with all conditions of this permit. Failure to comply with any permit condition is a violation of the Clean Water Act, Oregon Revised Statutes (ORS) 468B.025, and 40 Code of Federal Regulations (CFR) Section 122.41(a), and grounds for an enforcement action. Failure to comply is also grounds for the Department to modify, revoke, or deny renewal of a permit.

2. Penalties for Water Pollution and Permit Condition Violations
ORS 468.140 allows the Department to impose civil penalties up to $10,000 per day for violation of a term, condition, or requirement of a permit. Additionally 40 CFR 122.41 (A) provides that any person who violates any permit condition, term, or requirement may be subject to a federal civil penalty not to exceed $25,000 per day for each violation.

Under ORS 468.943 and 40 CFR 122.41(a), unlawful water pollution, if committed by a person with criminal negligence, is punishable by a fine of up to $25,000 imprisonment for not more than one year, or both. Each day on which a violation occurs or continues is a separately punishable offense.

Under ORS 468.946, a person who knowingly discharges, places, or causes to be placed any waste into the waters of the state or in a location where the waste is likely to escape into the waters of the state is subject to a Class B felony punishable by a fine not to exceed $200,000 and up to 10 years in prison. Additionally, under 40 CFR 122.41(a) any person who knowingly discharges, places, or causes to be placed any waste into the waters of the state or in a location where the waste is likely to escape into the waters of the state is subject to a federal civil penalty not to exceed $100,000 and up to 6 years in prison.

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

4. Duty to Reapply

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

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

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

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute
b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts
c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge

d. The permittee is identified as a Designated Management Agency or allocated a wasteload under a Total Maximum Daily Load (TMDL)
e. New information or regulations
f. Modification of compliance schedules
g. Requirements of permit reopener conditions
h. Correction of technical mistakes made in determining permit conditions
i. Determination that the permitted activity endangers human health or the environment
j. Other causes as specified in 40 CFR 122.62, 122.64, and 124.5

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

6. Toxic Pollutants

The permittee must comply with any applicable effluent standards or prohibitions established under Oregon Administrative Rules (OAR) 340-041-0033 for toxic pollutants within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

7. Property Rights and Other Legal Requirements

The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege, or authorize any injury to persons or property or invasion of any other private rights, or any infringement of federal, tribal, state, or local laws or regulations.

8. Permit References

Except for effluent standards or prohibitions established under OAR 340-041-0033 for toxic pollutants and standards for sewage sludge use or disposal established under Section 405(d) of the Clean Water Act, all rules and statutes referred to in this permit are those in effect on the date this permit is issued.

9. Permit Fees

The permittee must pay the fees required by Oregon Administrative Rules.

## SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance

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

2. Need to Halt or Reduce Activity Not a Defense

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

3. Bypass of Treatment Facilities

a. Definitions

(1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The term "bypass" does not apply if the diversion does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation or the diversion is due to nonuse of nonessential treatment units or processes at the treatment facility.

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

b. Prohibition of bypass.

(1) Bypass is prohibited unless:

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

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

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

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

c. Notice and request for bypass.

(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, a written notice must be submitted to the Department at least ten days before the date of the bypass.

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

4. <u>Upset</u>

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

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

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

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

(2) The permitted facility was at the time being properly operated;
(3) The permittee submitted notice of the upset as required in General Condition D.5, hereof (24-hour notice); and
(4) The permittee complied with any remedial measures required under General Condition A.3 hereof.

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

5. Treatment of Single Operational Upset

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

6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations

a. Definitions
(1) "Overflow" means the diversion and discharge of waste streams from any portion of the wastewater conveyance system including pump stations, through a designed overflow device or structure, other than discharges to the wastewater treatment facility.
(2) "Severe property damage" means substantial physical damage to property, damage to the conveyance system or pump station which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of an overflow.
(3) "Uncontrolled overflow" means the diversion of waste streams other than through a designed overflow device or structure, for example to overflowing manholes or overflowing into residences, commercial establishments, or industries that may be connected to a conveyance system.

b. Prohibition of storm related overflows. Storm related overflows of raw sewage are prohibited to waters of the State. However, the Environmental Quality Commission (EQC) recognizes that it is impossible to design and construct a conveyance system that will prevent overflows under all storm conditions. The State of Oregon has determined that all wastewater conveyance systems should be designed to transport storm events up to a specific size to the treatment facility. Therefore, such storm related overflows will not be considered a violation of this permit if:
(1) The permittee has conveyance and treatment facilities adequate to prevent overflows except during a storm event greater than the one-in-five-year, 24-hour duration storm from November 1 through May 21 and except during a storm event greater than the one-in-ten-year, 24-hour duration storm from May 22 through October 31. However, overflows during a storm event less than the one-in-five-year, 24-hour duration storm from November 1 through May 21 are also not permit violations if, the permittee had separate sanitary and storm sewers on January 10, 1996, had experienced sanitary sewer overflows due to inflow and infiltration problems, and has submitted an acceptable plan to the Department to address these sanitary sewer overflows by January 1, 2010;
(2) The permittee has provided the highest and best practicable treatment and/or control of wastes, activities, and flows and has properly operated the conveyance and treatment facilities in compliance with General Condition B.1.;
(3) The permittee has minimized the potential environmental and public health impacts from the overflow; and

(4) The permittee has properly maintained the capacity of the conveyance system.

c. Prohibition of other overflows. All overflows other than stormwater-related overflows (discussed in Schedule F, Section B, Condition 6.b.) are prohibited unless:
(1) Overflows were unavoidable to prevent an uncontrolled overflow, loss of life, personal injury, or severe property damage;
(2) There were no feasible alternatives to the overflows, such as the use of auxiliary pumping or conveyance systems, or maximization of conveyance system storage; and
(3) The overflows are the result of an upset as defined in General Condition B.4. and meeting all requirements of this condition.

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

e. Reporting required. Unless otherwise specified in writing by the Department, all overflows and uncontrolled overflows must be reported orally to the Department within 24 hours from the time the permittee becomes aware of the overflow. Reporting procedures are described in more detail in General Condition D.5. Reports concerning storm related overflows must include information about the amount and intensity of the rainfall event causing the overflow.

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

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

## SECTION C. MONITORING AND RECORDS

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

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

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

4. Penalties of Tampering

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

5. Reporting of Monitoring Results

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

6. Additional Monitoring by the Permittee

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

7. Averaging of Measurements

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

8. Retention of Records

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

9. Records Contents

Records of monitoring information must include:

a. The date, exact place, time, and methods of sampling or measurements;
b. The individual(s) who performed the sampling or measurements;
c. The date(s) analyses were performed;
d. The individual(s) who performed the analyses;
e. The analytical techniques or methods used; and
f. The results of such analyses.

10. Inspection and Entry

The permittee must allow the Department representative upon the presentation of credentials to:

a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;
b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;
c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit, and

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

## SECTION D. REPORTING REQUIREMENTS

1. Planned Changes
The permittee must comply with OAR chapter 340, division 52, "Review of Plans and Specifications" and 40 CFR Section 122.41(l) (1). Except where exempted under OAR chapter 340, division 52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers may be commenced until the plans and specifications are submitted to and approved by the Department. The permittee must give notice to the Department as soon as possible of any planned physical alternations or additions to the permitted facility.

2. Anticipated Noncompliance
The permittee must give advance notice to the Department of any planned changes in the permitted facility or activity that may result in noncompliance with permit requirements.

3. Transfers
This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and the rules of the Commission. No permit may be transferred to a third party without prior written approval from the Department. The Department may require modification, revocation, and reissuance of the permit to change the name of the permittee and incorporate such other requirements as may be necessary under the Clean Water Act (see 40 CFR Section 122.61; in some cases, modification or revocation and reissuance is mandatory).. The permittee must notify the Department when a transfer of property interest takes place.

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

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

A written submission must also be provided within 5 days of the time the permittee becomes aware of the circumstances. Pursuant to ORS 468.959 (3) (a), if the permittee is establishing an affirmative defense of upset or bypass to any offense under ORS 468.922 to 468.946, delivered written notice must be made to the Department or other agency with regulatory jurisdiction within 4 (four) calendar days of the time the permittee becomes aware of the circumstances. The written submission must contain:

a. A description of the noncompliance and its cause;
b. The period of noncompliance, including exact dates and times;
c. The estimated time noncompliance is expected to continue if it has not been corrected;
d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and
e. Public notification steps taken, pursuant to General Condition B.7

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

f. Any unanticipated bypass that exceeds any effluent limitation in this permit;
g. Any upset that exceeds any effluent limitation in this permit;
h. Violation of maximum daily discharge limitation for any of the pollutants listed by the Department in this permit; and
i. Any noncompliance that may endanger human health or the environment.

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

6. Other Noncompliance

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

a. A description of the noncompliance and its cause;
b. The period of noncompliance, including exact dates and times;
c. The estimated time noncompliance is expected to continue if it has not been corrected; and
d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

7. Duty to Provide Information

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

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

8. Signatory Requirements

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

9. Falsification of Information

Under ORS 468.953, any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or noncompliance, is subject to a Class C felony punishable by a fine not to exceed $100,000 per violation and up to 5 years in prison. Additionally, according to 40 CFR 122.41(k)(2), any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit including monitoring reports or reports of compliance or non-compliance shall, upon conviction, be punished by a federal civil penalty not to exceed $10,000 per violation, or by imprisonment for not more than 6 months per violation, or by both.

10. Changes to Indirect Dischargers

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

a. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of the Clean Water Act if it were directly discharging those pollutants and;
b. Any substantial change in the volume or character of pollutants being introduced into the POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.
c. For the purposes of this paragraph, adequate notice shall include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

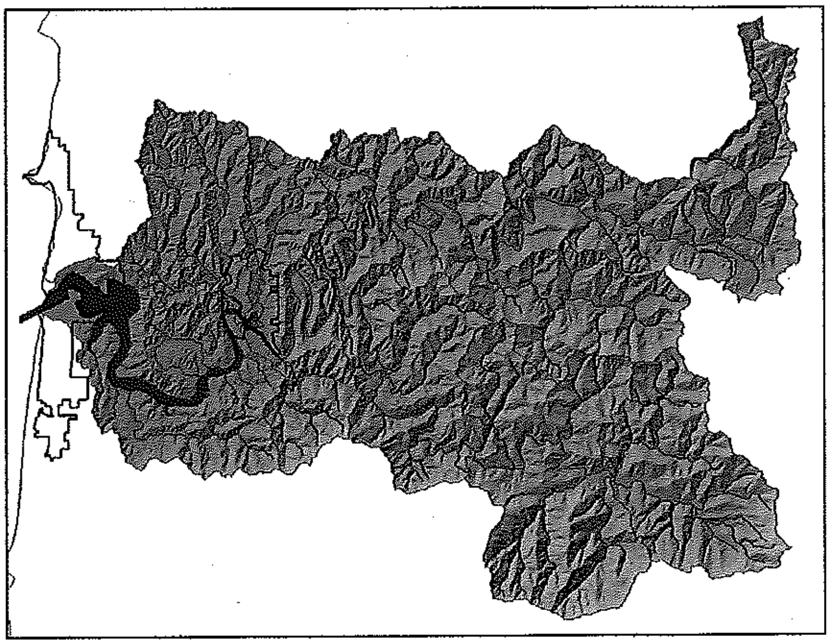
## SECTION E. DEFINITIONS

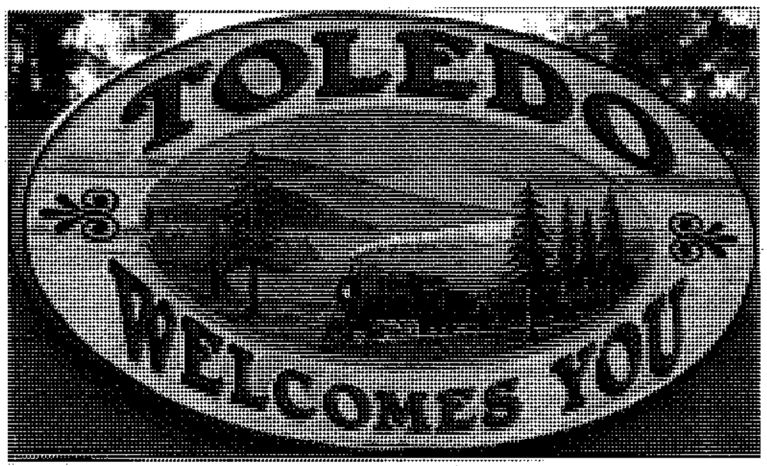
1. *BOD* means five-day biochemical oxygen demand.
2. *CBOD* means five day carbonaceous biochemical oxygen demand
3. *TSS* means total suspended solids.
4. "*Bacteria*" includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.
5. *FC* means fecal coliform bacteria.
6. *Total residual chlorine* means combined chlorine forms plus free residual chlorine
7. *Technology based permit effluent limitations* means technology-based treatment requirements as defined in 40 CFR Section 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR Chapter 340, Division 41.
8. *mg/l* means milligrams per liter.
9. *kg* means kilograms.
10. $m^3/d$ means cubic meters per day.
11. *MGD* means million gallons per day.
12. 24-hour *Composite sample* means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow. The sample must be collected and stored in accordance with 40 CFR part 136.
13. *Grab sample* means an individual discrete sample collected over a period of time not to exceed 15 minutes.
14. *Quarter* means January through March, April through June, July through September, or October through December.
15. *Month* means calendar month.
16. *Week* means a calendar week of Sunday through Saturday.
17. *POTW* means a publicly owned treatment works

# APPENDIX B

APPENDIX B

# Near-field Modeling of the City of Toledo Wastewater Discharge into the Yaquina River






by
Scott A. Wells, Ph.D., P.E.,
and
Robert L. Annear Jr., M.S.

Prepared for City of Toledo
Project Manager: Herbert Jennings

August 2005

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## Introduction

The City of Toledo currently discharges wastewater from their treatment facility to Yaquina River approximately 12.7 miles (20.4 km) upstream from the ocean. Yaquina Bay is located on the coast south and west of the City of Toledo, Oregon and the bay is surrounded by the City of Newport, Oregon as shown in Figure 1 and Figure 2. The discharge location is downstream of the Butler Road Bridge, as shown in Figure 3. A near-field mixing zone analysis was required to determine the water quality impacts of this discharge on the Yaquina River.

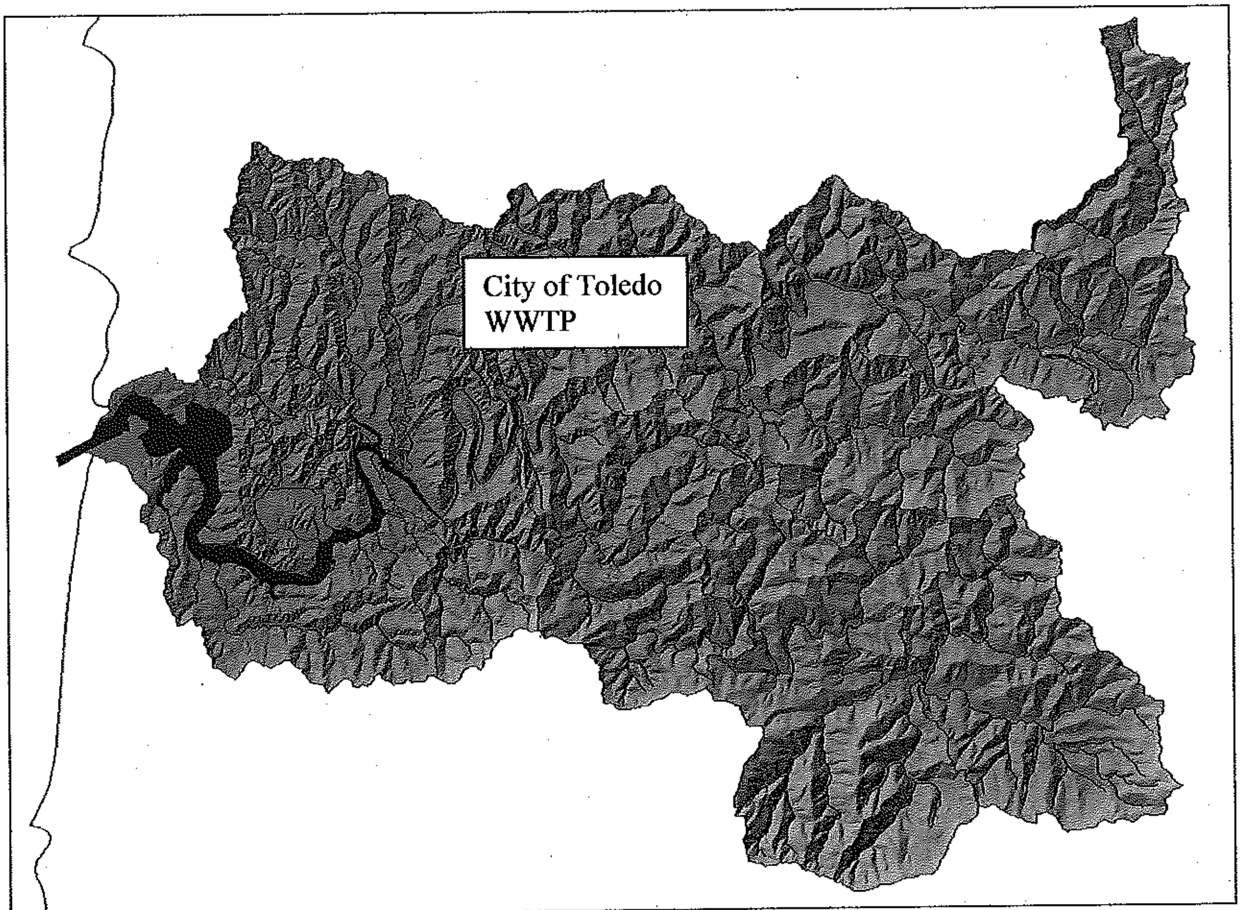


**Figure 1: City of Toledo WWTP in the Yaquina River basin on the Oregon Coast**

This project incorporated the following steps:

1. Obtain boundary condition and bathymetric data
   a. Bathymetric data for the Yaquina River in the vicinity of the Toledo STP discharge
   b. Obtain stage data at the City of Toledo (or computed stage information)
   c. Yaquina River flow and temperature data
   d. Toledo sewage treatment plant (STP) discharge data – flow, chlorine, temperature, ammonia
   e. Acquire and compile field data from earlier field studies in Yaquina Bay

APPENDIX B

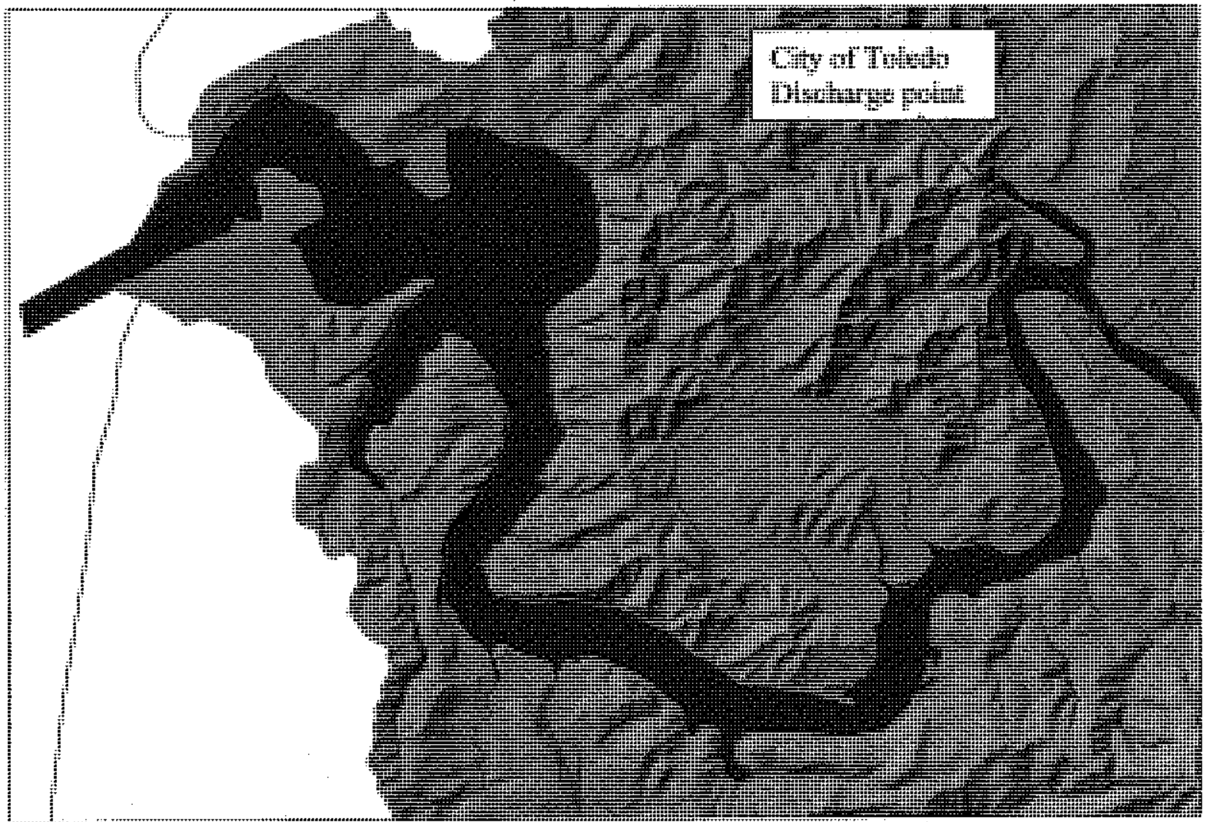


Figure 2: City of Toledo WWTP effluent discharge point location


Figure 3: Location of discharge point for City of Toledo upstream of the Butler Road Bridge.

2. Review NPDES permit conditions and analyze boundary condition data
    a. Compute 7Q10 flows and develop expected critical flow and stage conditions for evaluating the discharge into the Yaquina River
3. Use a near-field model to predict dilution at the zone of initial dilution and at the edge of the mixing zone
4. Write a technical memorandum summarizing findings and analysis of toxicity issues within and at the edge of the mixing zone
5. Collaborate with City of Toledo (and Oregon DEQ) in obtaining information and coordinating the modeling needs as required by Oregon DEQ.

This technical report includes the following sections:

- Information on the Toledo WWTP discharge flow rates and effluent concentrations
- Bathymetric information in the vicinity of the outfall
- Resource maps for the Yaquina Bay estuary
- Historical information on tidal height, temperature and other water quality parameters taken in Yaquina Bay
- Statistical analysis of Yaquina River flow rates in order to compute 7Q10 and 1Q10 flows
- Use of the CE-QUAL-W2 model (Cole and Wells, 2004) in predicting tidal conditions at the outfall
- Use of an analytical near field mixing model and CORMIX to predict dilution at the edge of the mixing zone
- Discussion of model results in terms of toxicity impacts on the Yaquina River

## Background Data

There have been several water quality field studies performed in the Yaquina River system. One was performed by Furfari (1985) where water quality data were obtained during several months in 1984. A summary of much of these data are included in Appendix IV. Included in this report was a far field dye study release. In 1991 a hydrographic survey of Yaquina Bay was performed where the far-field dye plume from the City of Toledo wastewater treatment plant was tracked through several tidal cycles (Unknown, 1992). These studies were primarily focused on the impact of bacteria from the City of Toledo on the shellfish harvesting in Yaquina Bay.

The background data reviewed in this section includes:

- City of Toledo outfall characteristics
- City of Toledo outflow flow rates
- City of Toledo effluent concentrations and temperatures
- Yaquina Bay water level data
- Yaquina Bay temperature data
- Yaquina River flow rates
- Morphology of Yaquina River in the vicinity of the outfall
- Resource maps of discharges, shellfish areas, water quality monitoring sites and beaches

### City of Toledo Outfall

The City of Toledo outfall is essentially a side-discharge pipe on the right bank of the Yaquina River at the Butler Bridge at RM 10.2. Detailed information on the outfall is shown in Appendix I with pictures of the river and outfall shown in Appendix II. The Oregon DEQ mixing zone is defined as a 100 ft radius from the discharge point.

### Wastewater Treatment Plant Flow Rates

According to Oregon DEQ, the City of Toledo is classified as a minor discharge with an average dry weather flow (ADWF) of 0.73 MGD. The plant has a maximum hydraulic capacity of 3.5 MGD. Typical plant flow though is less than 0.5 MGD during the dry months with peaks almost as high as 3.5 MGD during the rainy season when there is infiltration. Plant effluent flow rate for the entire year of 2002 is shown in Figure 4, and plant flow rate from January 1 through March 25, 2005 is shown in Figure 5. Descriptive statistics of these flows are shown in Table 1.

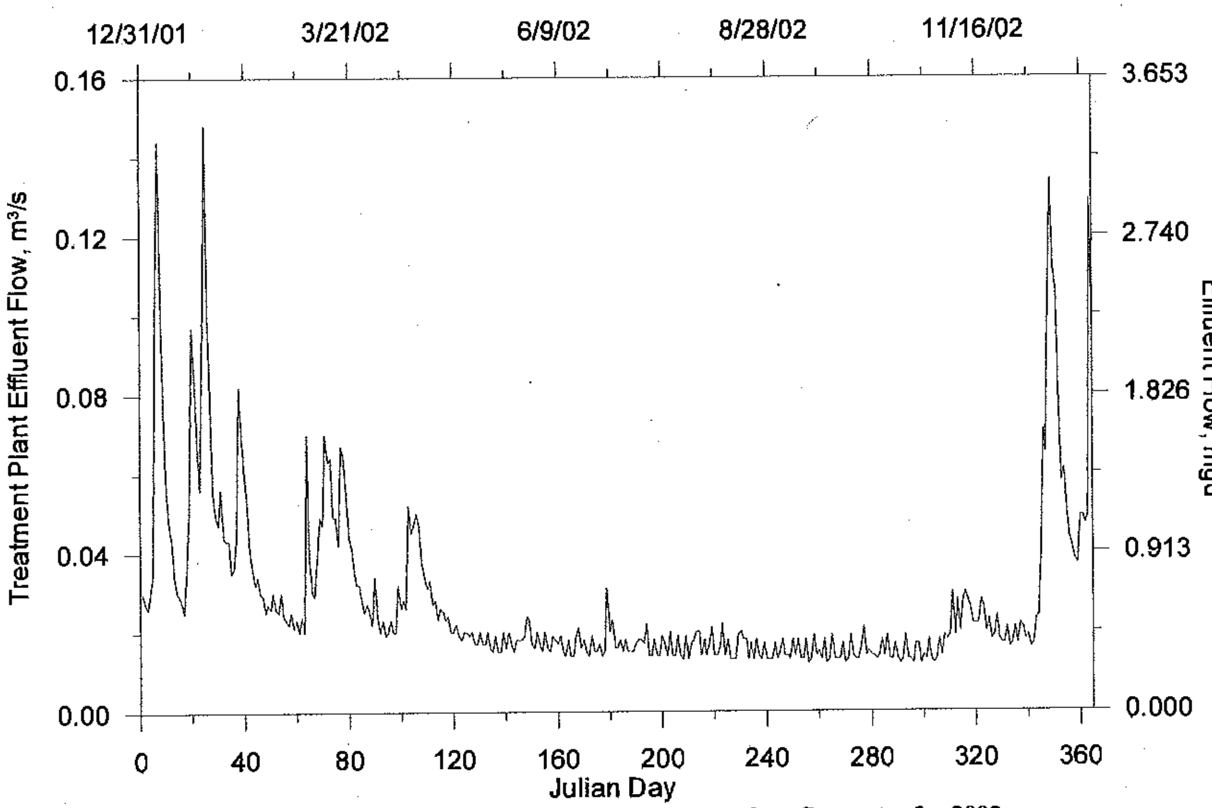


**Figure 4: City of Toledo wastewater treatment plant flow rates for 2002.**

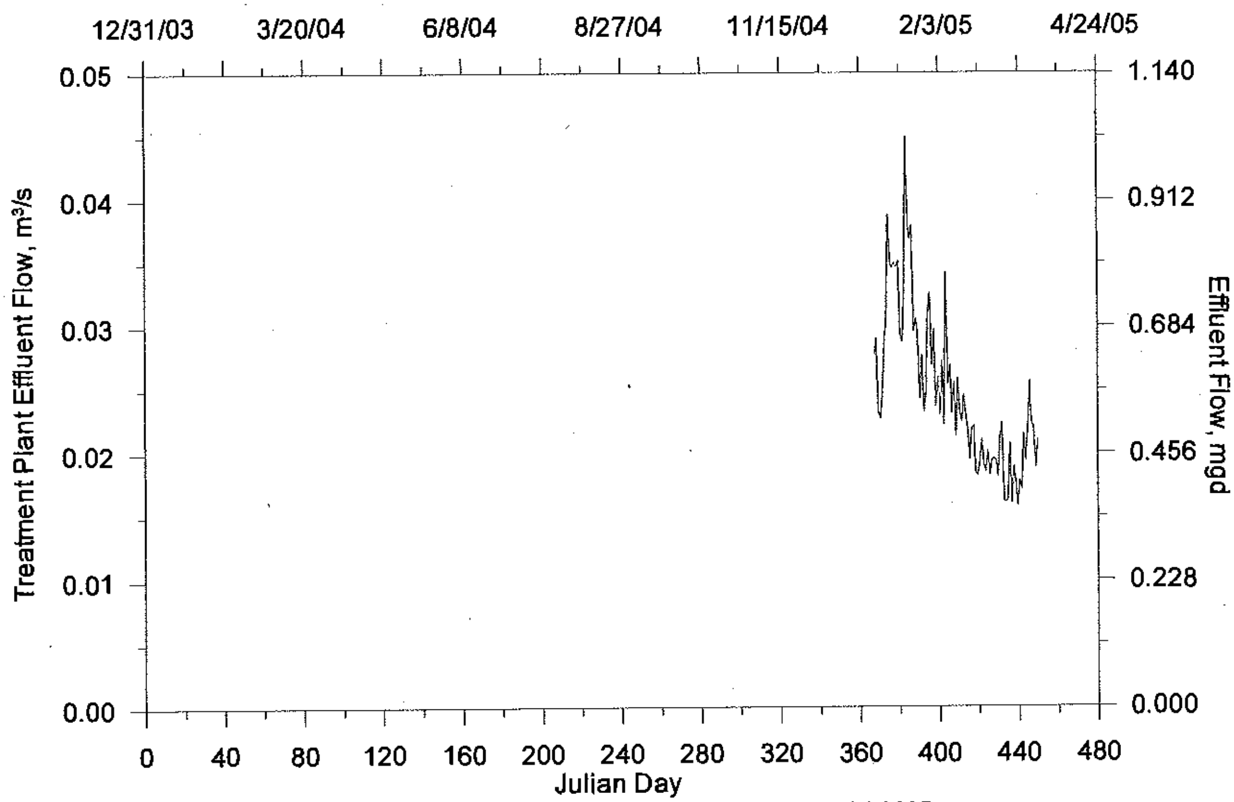


**Figure 5: City of Toledo WWTP effluent flow, 2004-2005**

**Table 1: Summary statistics for City of Toledo effluent flow for January 1, 2005 through March 25, 2005 and for the 2002 year.**

| Statistic | 1/1/05-3/25/05 Flow rate, MGD | 1/1/02-12/31/02 Flow rate, MGD |
|---|---|---|
| Mean | 0.567 | 0.645 |
| Standard Error | 0.016 | 0.026 |
| Median | 0.528 | 0.445 |
| Mode | 0.614 | 0.395 |
| Standard Deviation | 0.145 | 0.503 |
| Sample Variance | 0.021 | 0.253 |
| Kurtosis | 0.222 | 8.328 |
| Skewness | 0.843 | 2.678 |
| Range | 0.663 | 3.113 |
| Minimum | 0.362 | 0.274 |
| Maximum | 1.025 | 3.387 |
| Sum | 47.023 | 235.325 |
| Count | 83 | 365 |

## *Wastewater Treatment Plant Effluent Quality*

According to the NPDES permit for the City of Toledo, they are required to comply with acute and chronic toxicity standards for toxic substances within and at the edge of the mixing zone. Also, the permit recognizes that the discharge is in a region of shellfish harvesting and bacteria requirements in the effluent were stated as a monthly geometric mean of 100 Fecal coliform/100 ml and a weekly maximum geometric mean of 200/100 ml. Temperature for point source discharges is found within the Oregon DEQ (DEQ, 2004) water quality criteria where the well-mixed discharge must not contribute more than 0.3°C excess temperature to the Yaquina River.

Records of effluent concentrations of ammonia concentration as N, temperature, pH, chlorine residual, and coliform bacteria between January 2004 and March 31, 2005 were provided. Figure 6 shows a time series plot of the effluent temperature in 2004 and early 2005. Table 2 lists the monthly average effluent temperatures from the 2004-2005 data set and Figure 7 shows the same monthly averages as a bar chart. The two figures and table show the effluent temperature show a seasonal trend of increasing temperatures into late summer and then decreasing as the year ends.

Figure 8 shows a times series plot pH through 2004 and part of 2005. The figure shows there is an increase in pH from winter into summer and then the pH decreases moving into fall and winter again. Figure 9 show the ammonia concentration for 2002 and Figure 10 shows the ammonia concentration in 2005-2005. The two figures indicate there was much higher ammonia concentration in 2002 than in 2004.

Figure 11 shows a time series plot of the residual chlorine concentration in the effluent in 2004-2005 and Figure 12 shows a plot of the fecal coliform concentration in the effluent over the same time period. Statistics of these water quality constituents between January 1, 2004 and March 25, 2005 are shown in Table 3.

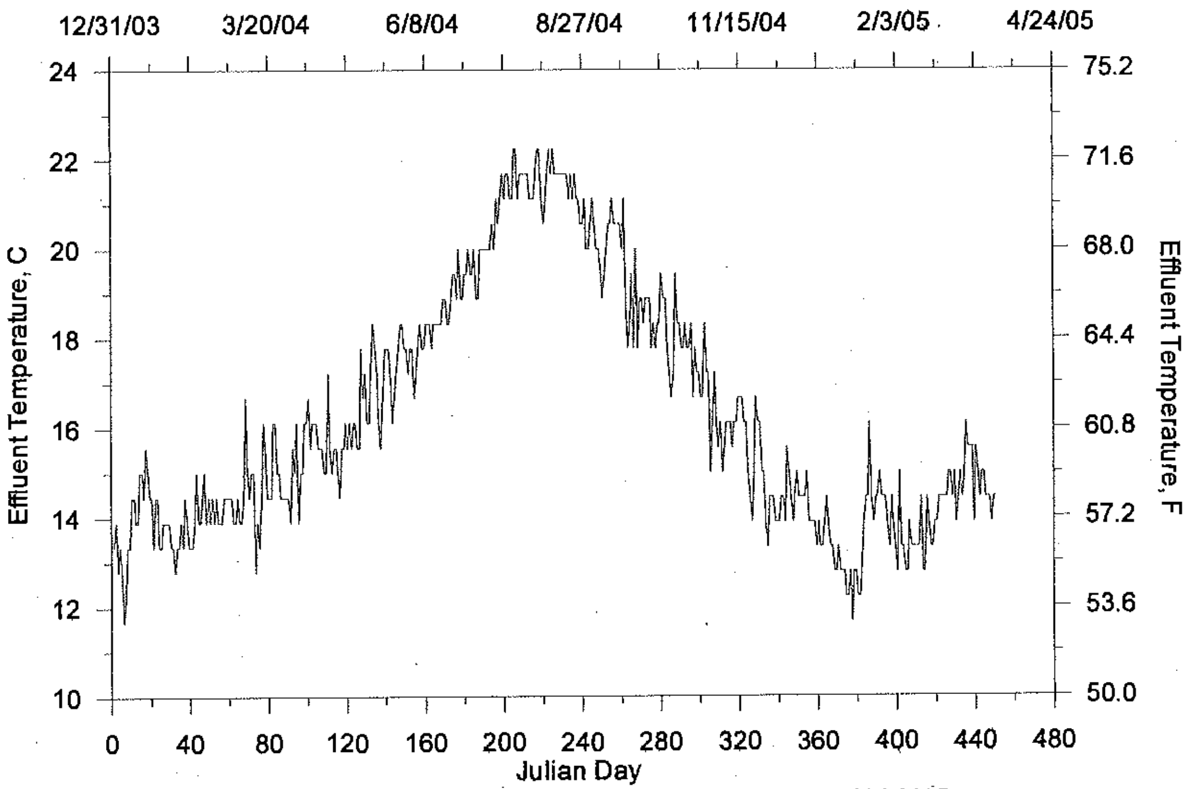


**Figure 6: City of Toledo WWTP effluent temperature, 2004-2005**

**Table 2: Monthly average of effluent temperature data from 2004 and 2005.**

| Month | Ave. Temp, C | Month | Ave. Temp, C |
|---|---|---|---|
| Jan | 13.7 | Jul | 20.8 |
| Feb | 13.8 | Aug | 21.3 |
| Mar | 14.7 | Sep | 19.6 |
| Apr | 15.6 | Oct | 17.8 |
| May | 17.0 | Nov | 15.6 |
| Jun | 18.5 | Dec | 14.2 |

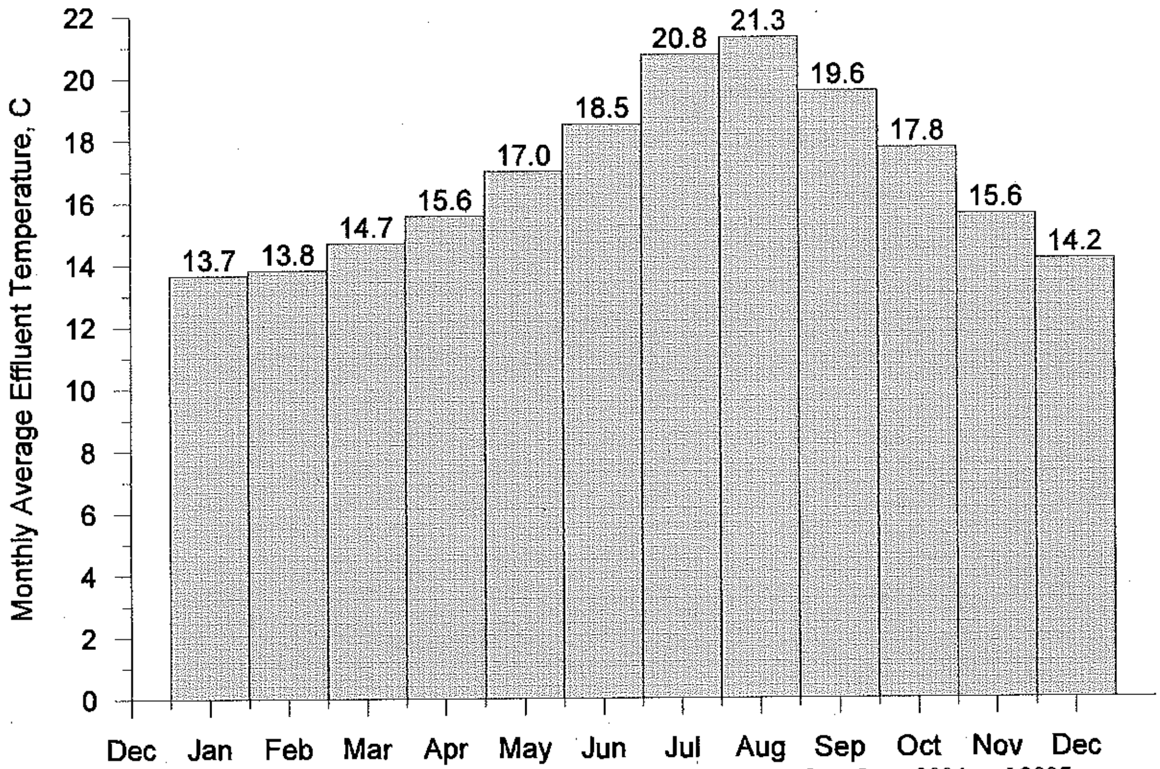


**Figure 7: Monthly average of City of Toledo effluent temperature data from 2004 and 2005.**

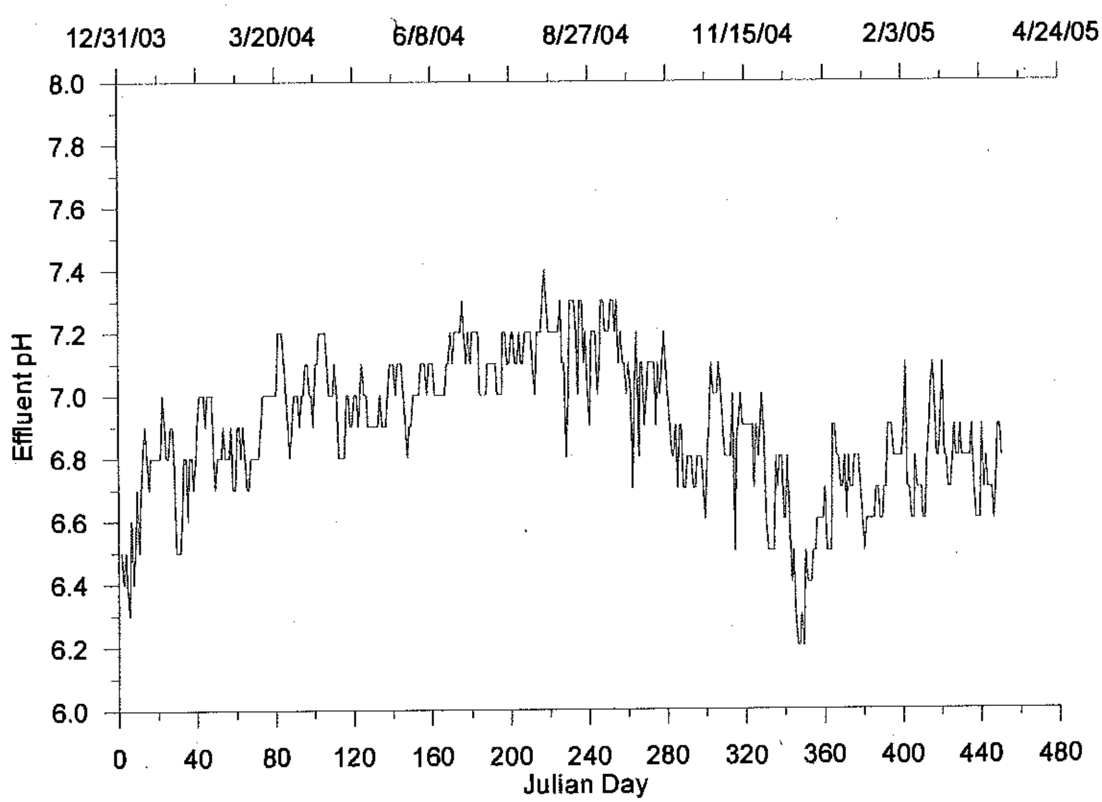


**Figure 8: City of Toledo WWTP effluent pH, 2004-2005**

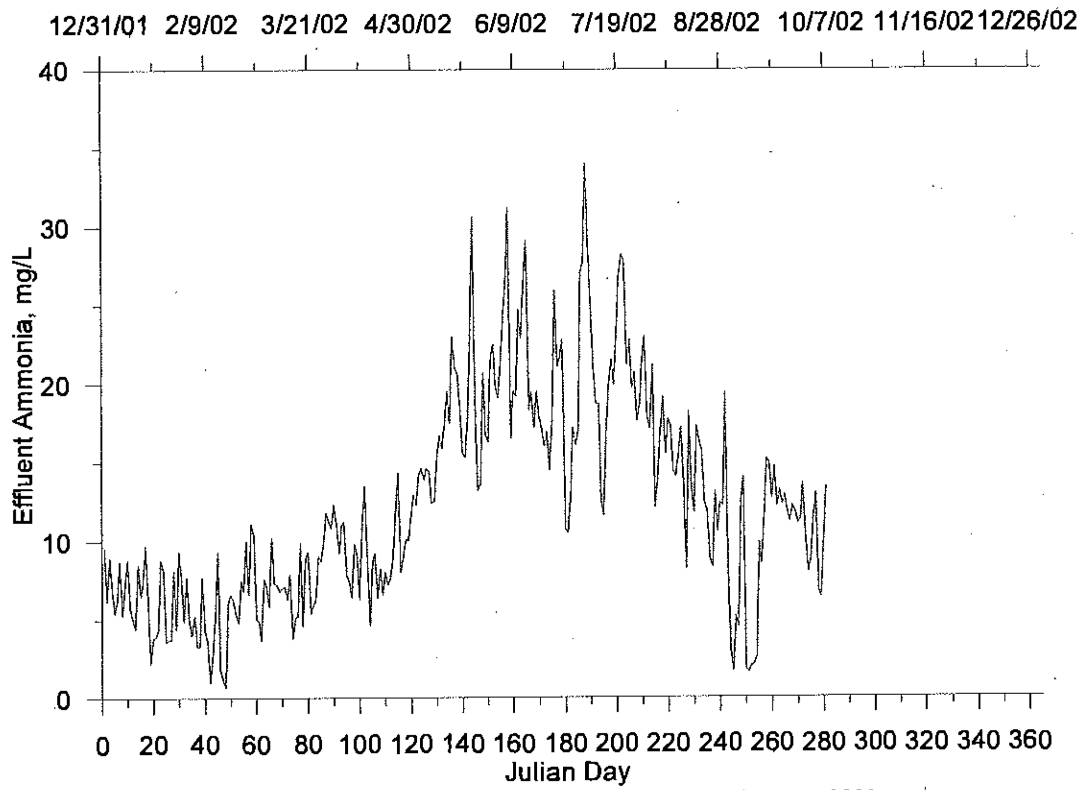


**Figure 9: City of Toledo WWTP effluent ammonia data, 2002**

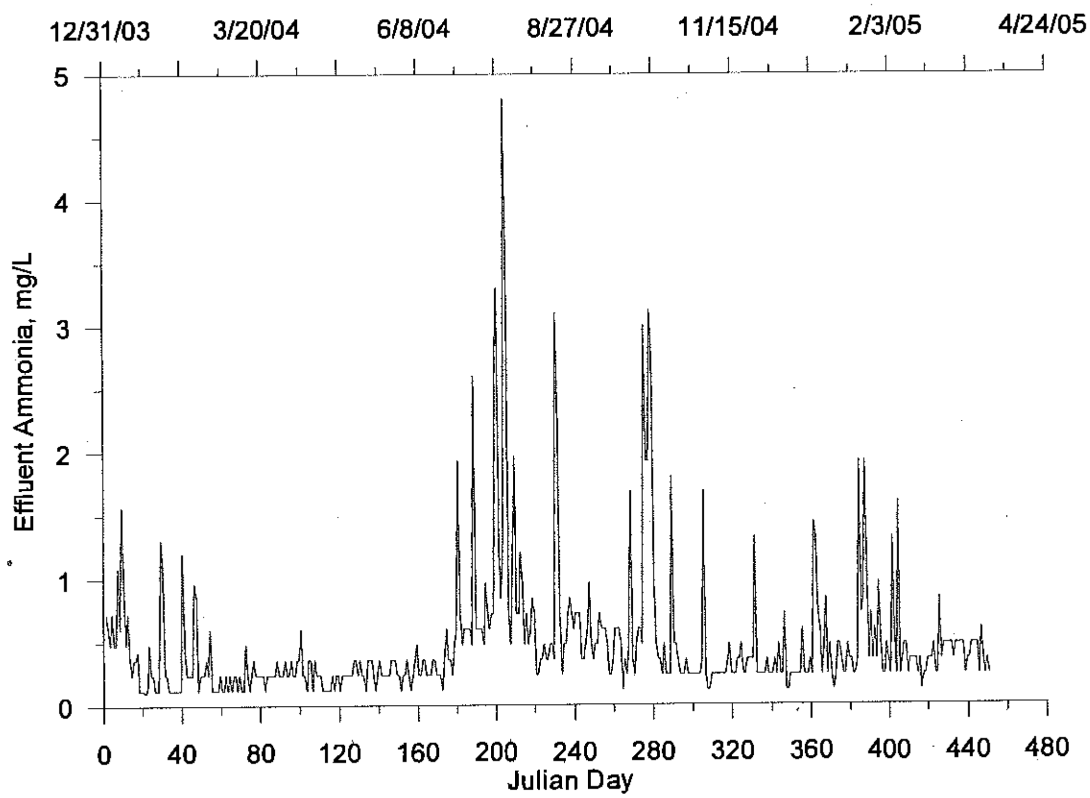


**Figure 10: City of Toledo WWTP effluent ammonia, 2004-2005**

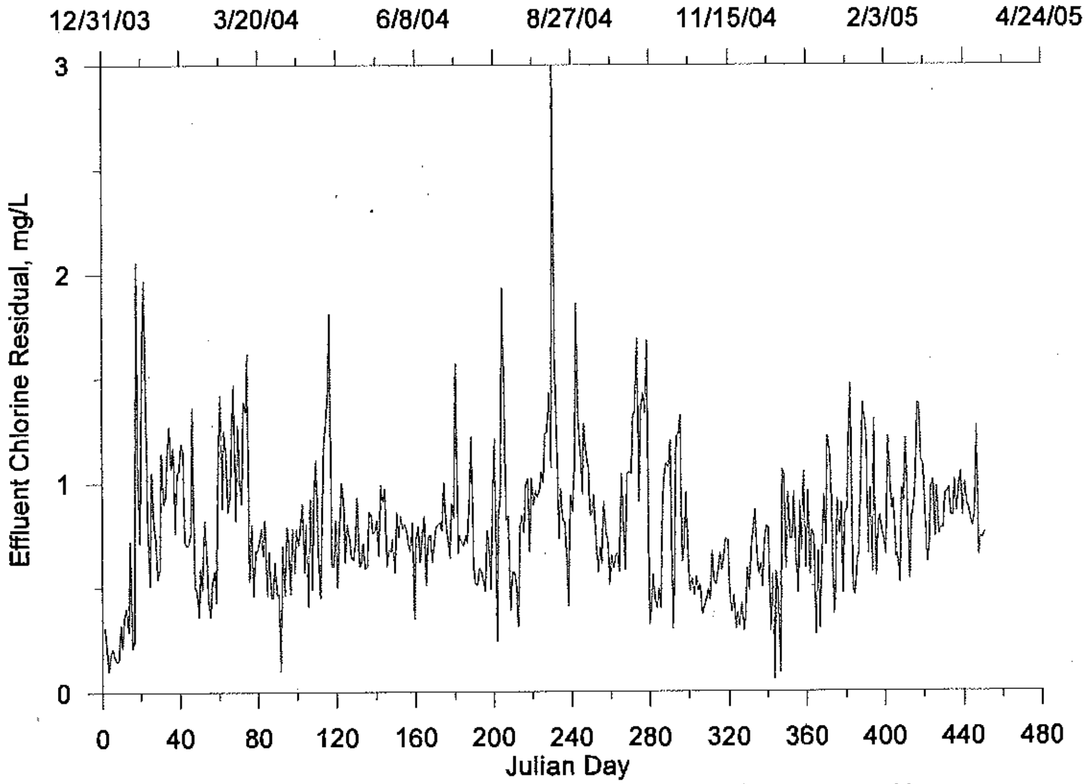


**Figure 11: City of Toledo WWTP effluent residual chlorine, 2004-2005**

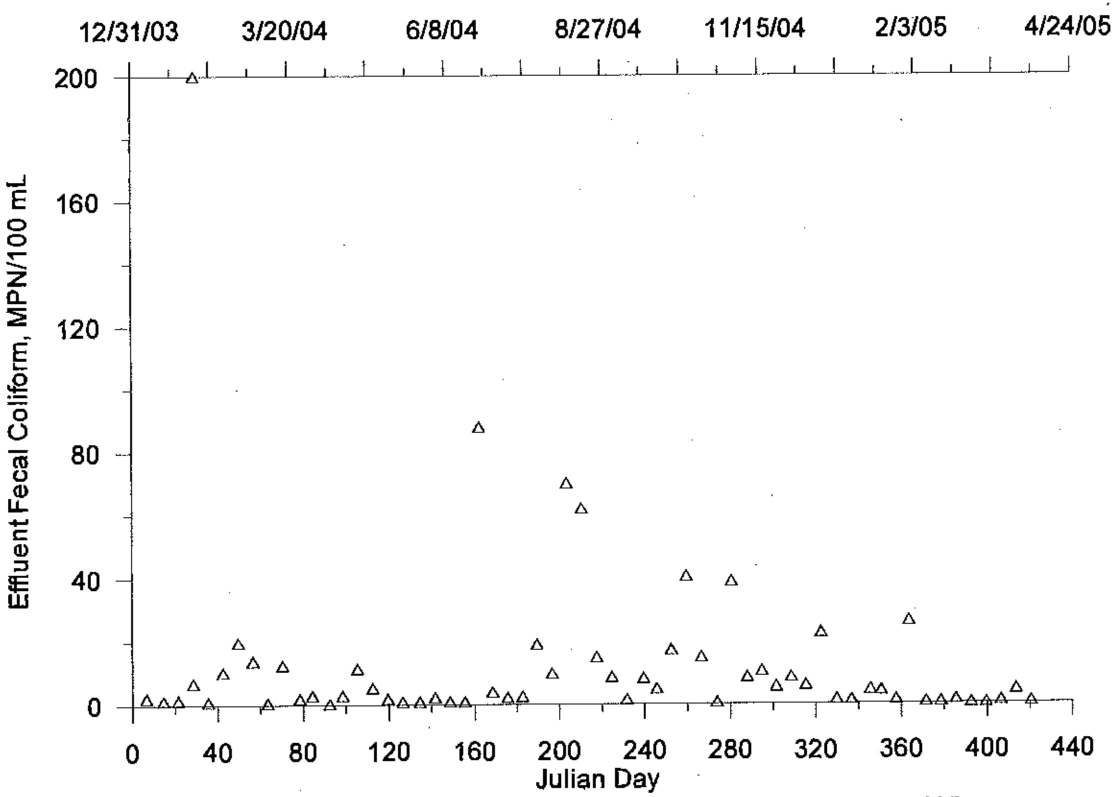


**Figure 12: City of Toledo WWTP effluent fecal coliform count, 2004-2005**

**Table 3: Summary statistics for the City of Toledo effluent ammonia, pH, temperature, Chlorine residual, fecal coliform between January 1, 2004 and March 25, 2005.**

| Statistic | Ammonia as N, mg/l | pH | Temperature, F | Cl Residual, mg/l | Fecal coliform, #/100 ml |
|---|---|---|---|---|---|
| Mean | 0.48 | 6.9 | 61.44 | 0.79 | 13.5 |
| Standard Error | 0.02 | 0.0 | 0.23 | 0.02 | 3.8 |
| Median | 0.36 | 6.9 | 60.00 | 0.76 | 4.3 |
| Mode | 0.24 | 6.8 | 58.00 | 0.84 | 1.0 |
| Standard Deviation | 0.52 | 0.2 | 4.90 | 0.34 | 29.6 |
| Sample Variance | 0.27 | 0.1 | 24.05 | 0.11 | 877.5 |
| Kurtosis | 20.21 | -0.09 | -0.85 | 4.48 | 26.9 |
| Skewness | 3.98 | -0.41 | 0.58 | 1.21 | 4.8 |
| Range | 4.7 | 1.2 | 19.00 | 2.94 | 199.7 |
| Minimum | 0.1 | 6.2 | 53 | 0.06 | 0.3 |
| Maximum | 4.8 | 7.4 | 72 | 3.00 | 200.0 |
| Sum | 216.72 | 3104.18 | 27646 | 357.07 | 824.2 |
| Count | 450 | 450 | 450 | 450 | 61 |

## *Water Level Elevation Frequency Analysis*

There are several sites in Yaquina Bay and River where the water level is currently monitored or has been in the past through the Center for Operational Oceanographic Products and Services, National Ocean Service, which is part of NOAA. There are four water level monitoring sites in Yaquina Bay with only one currently monitoring data (South Beach). Figure 13 shows a map of Yaquina Bay and the location of these monitoring sites. The sites which no longer have ongoing data collection are related to a west coast reference site in Crescent City, CA. Table 4 lists the water level sites, the years of data and the status on whether the gages are currently active.

The water level data at the Toledo, OR site were calculated using the NOAA-NOS methodology (http://www.co-ops.nos.noaa.gov/tide_pred.html) to correct the tidal predictions from Crescent City, CA to Toledo, OR. These tidal predictions only included the high and low tides and not the entire tidal cycle. The tidal corrections from Crescent City, CA to Toledo, OR are:

| | | |
|---|---|---|
| High tides: | Time: +89 min | Heights*1.17 |
| Low tides: | Time: +99 min | Heights*0.92 |

Once the tidal predictions from Crescent City, Ca from 1991 to 2005 were corrected, Toledo, OR (MLLW) the tidal predictions were converted to a vertical datum of NGVD29 using the equation: NGVD29 = MLLW – 1.259 ft.

Frequency occurrence plots of the slack tides were examined over all of the tidal predictions from 1991 to 2005 and for only the tidal predictions in September of each year. Figure 14 shows frequency plots for low water and low low water. Figure 15 shows frequency plots for high water and high high water at Toledo, OR. Figure 16 shows frequency plots for low water and low low water in the month of

September and Figure 17 shows frequency plots for high water and high high water in September. The figures show that each slack tide has a wide range of elevations over a year. When the water level frequency for just September is considered the water level range for each slack tide is smaller than the rest of the year. This is to be expected when considering the reduced Yaquina River flow in September.

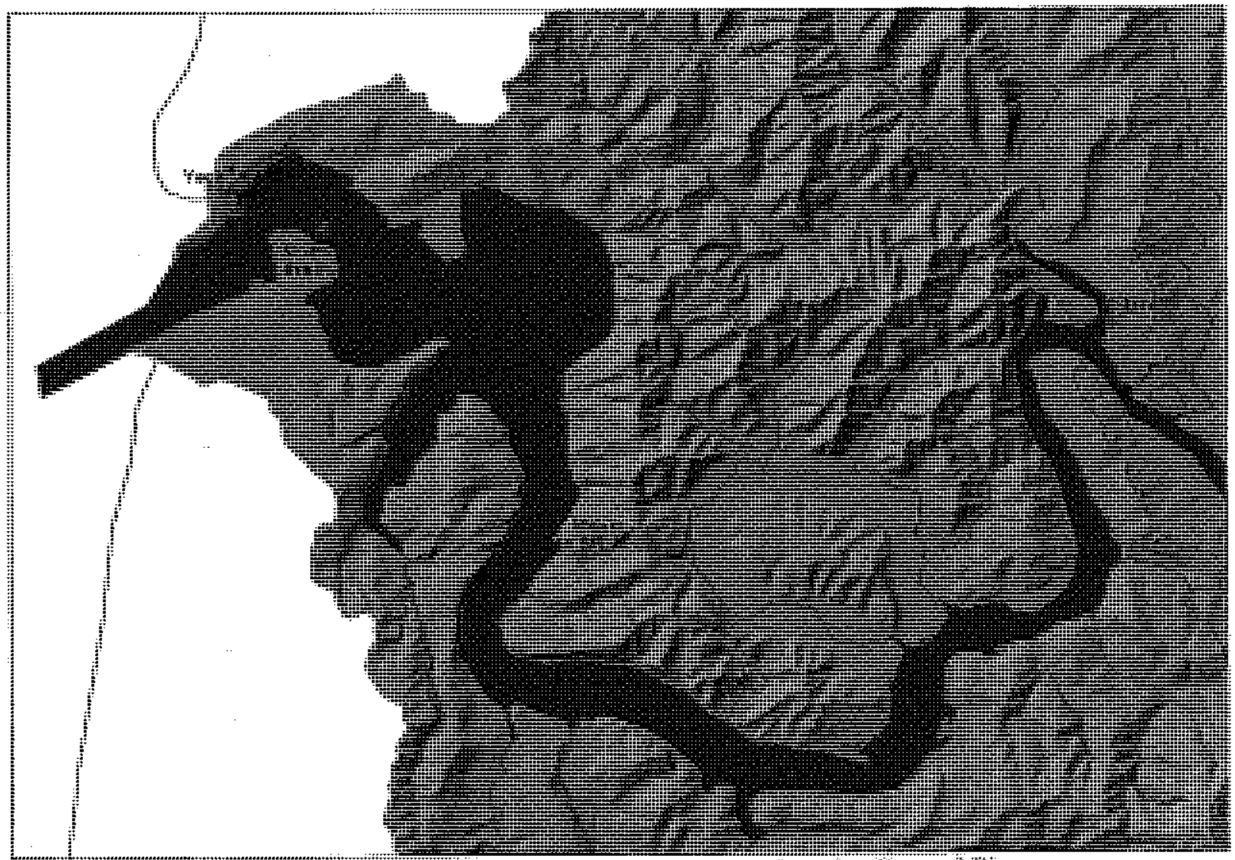

Figure 13: Historical and active water level gages in Yaquina Bay and River

**Table 4: Water level sites and extent of data for Yaquina Bay and River**

| Site ID | Site Description | Years of data | Comment |
|---|---|---|---|
| 9435380 | South Beach, Yaquina River | 1991 to 2005 | Current, Active gage |
| 9435362 | Toledo, OR | 1982 | Not current, Active gage |
| 9435308 | Weiser Point, Yaquina River | 1982 | Not current, Active gage |
| 9435385 | Yaquina USCG Station, Newport, OR | 1982 | Not current, Active gage |
| 9419750 | Crescent City, CA | 1991 to 2005 | Current, Active gage |

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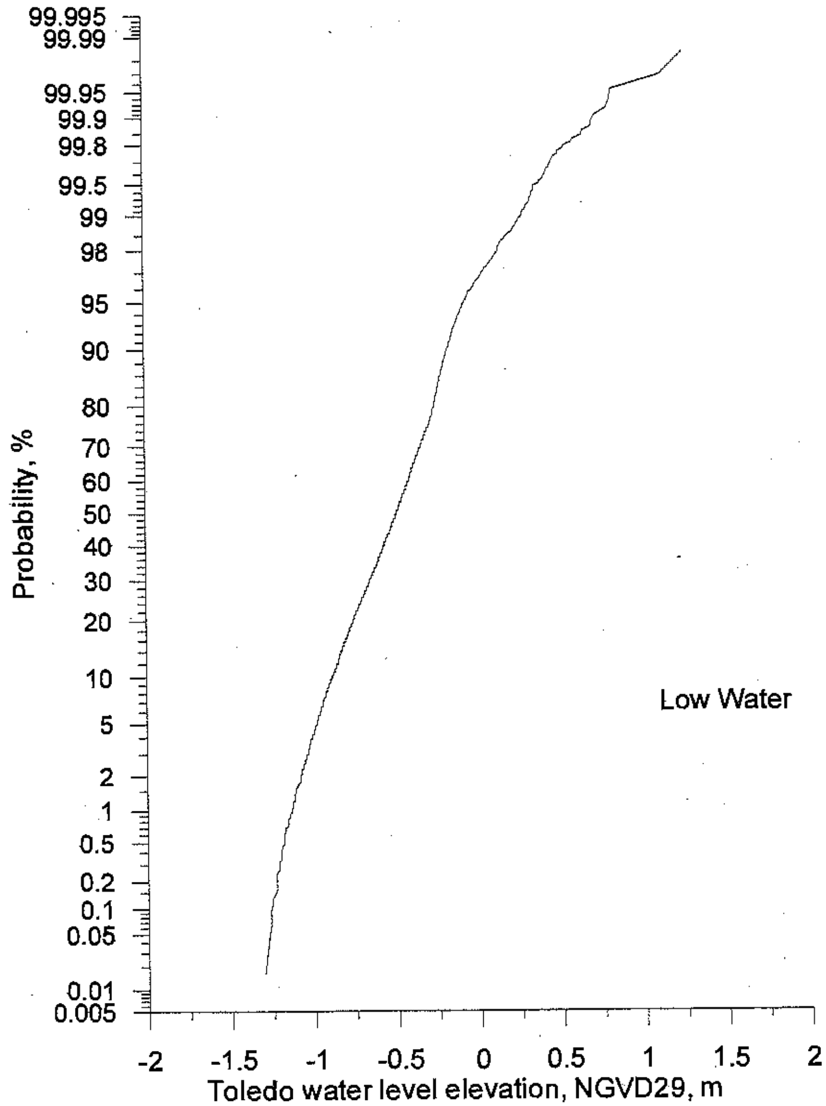


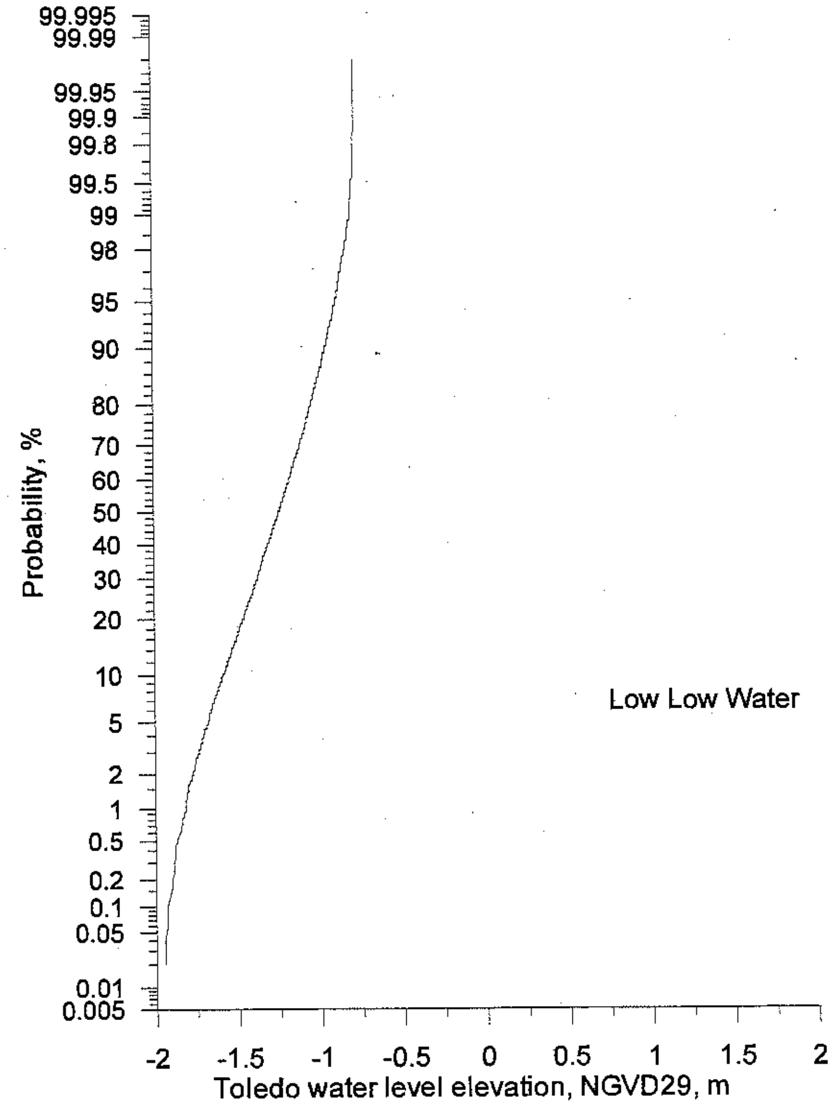


**Figure 14: Calculated low water and low low water elevation frequencies at Toledo, OR**

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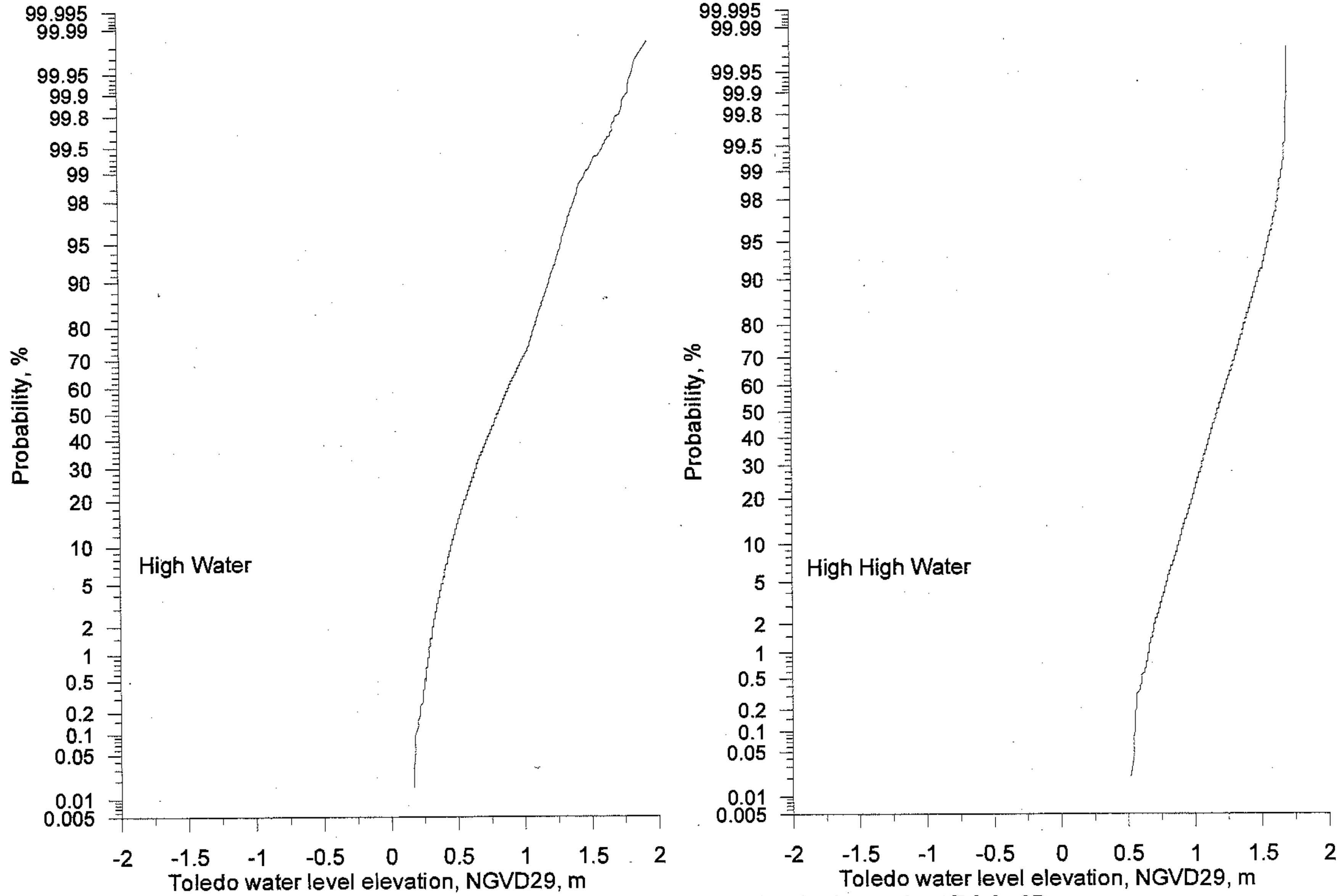


Figure 15: Calculated high water and high high water elevation frequencies at Toledo, OR

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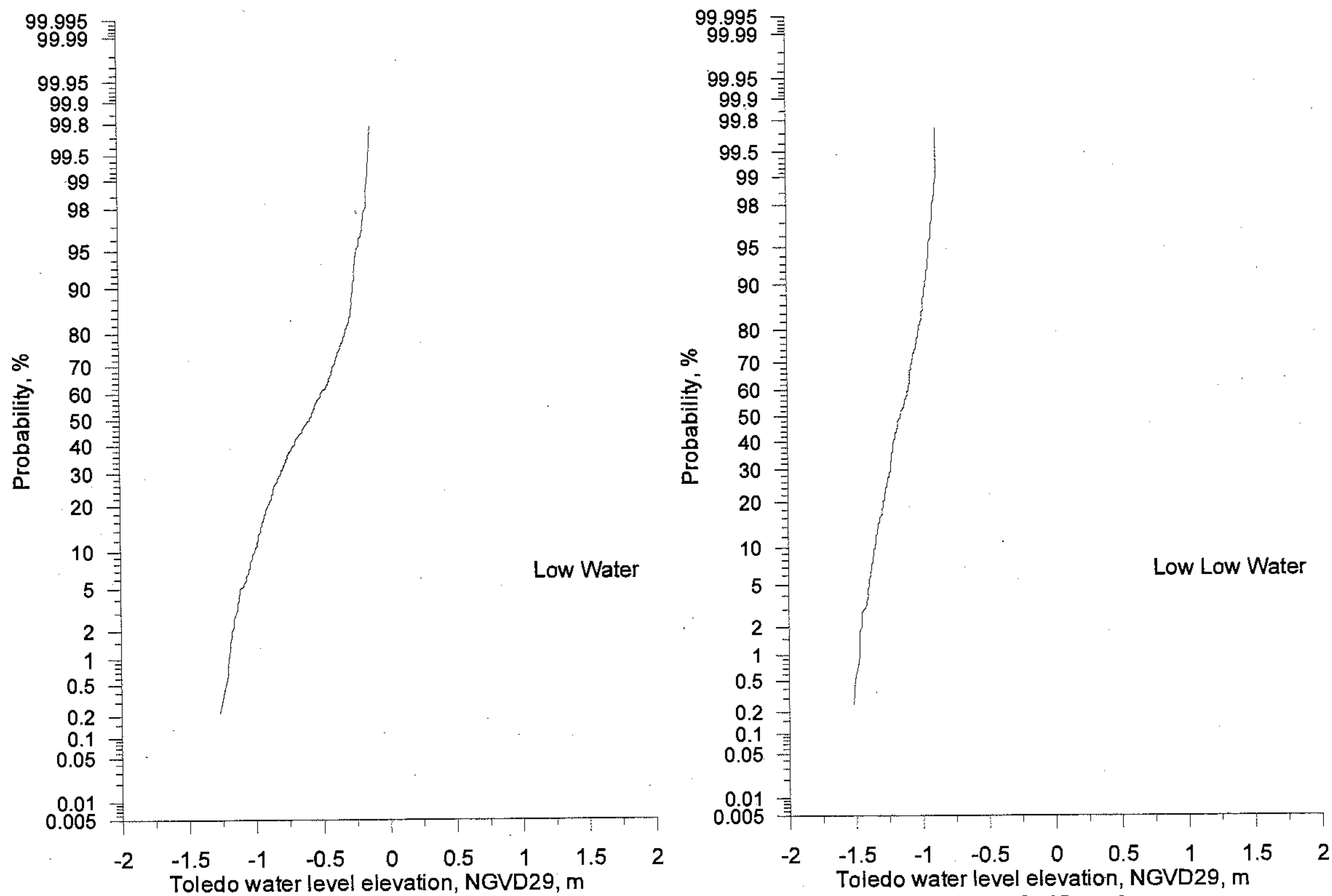


**Figure 16: Calculated low water and low low water elevation frequencies at Toledo, OR for the month of September**

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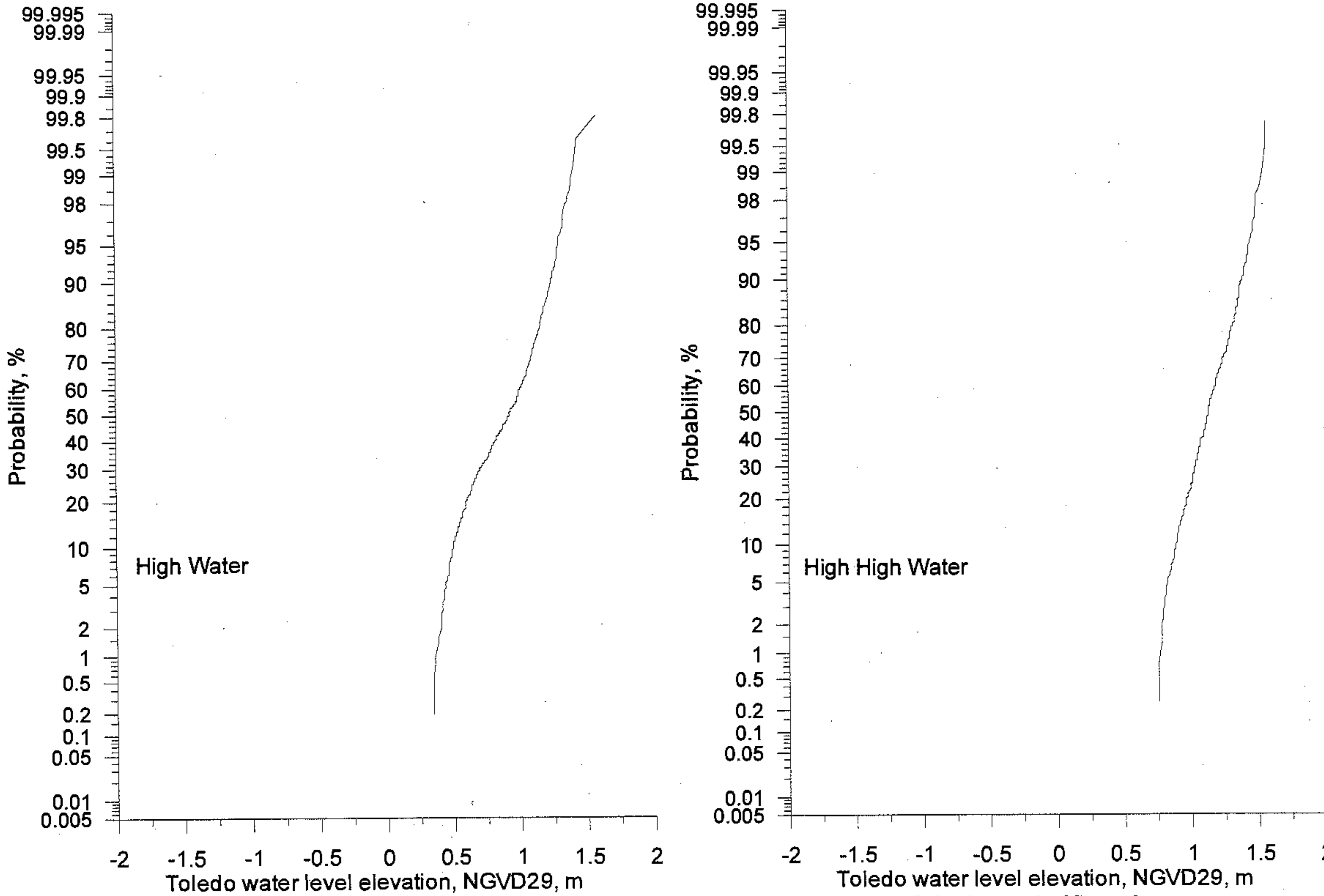


**Figure 17: Calculated high water and high high water elevation frequencies at Toledo, OR for the month of September**

### *Water Temperature Frequency Analysis*

A water temperature frequency analysis was conducted using several monitoring sites in the Yaquina River and Bay that were available. Figure 18 shows the temperature monitoring sites in Yaquina River and Bay. Figure 19 shows a map of the area around the City of Toledo outfall and the nearest temperature monitoring sites upstream and downstream. Primarily there are three monitoring two monitored by ODEQ and a third monitored by Oregon State University as part of the NOAA tidal gage network. Table 5 lists the three monitoring sites and the extent of data available for each site.

Table 6 lists the number of data points in each calendar month over the date ranges listed in Table 5.

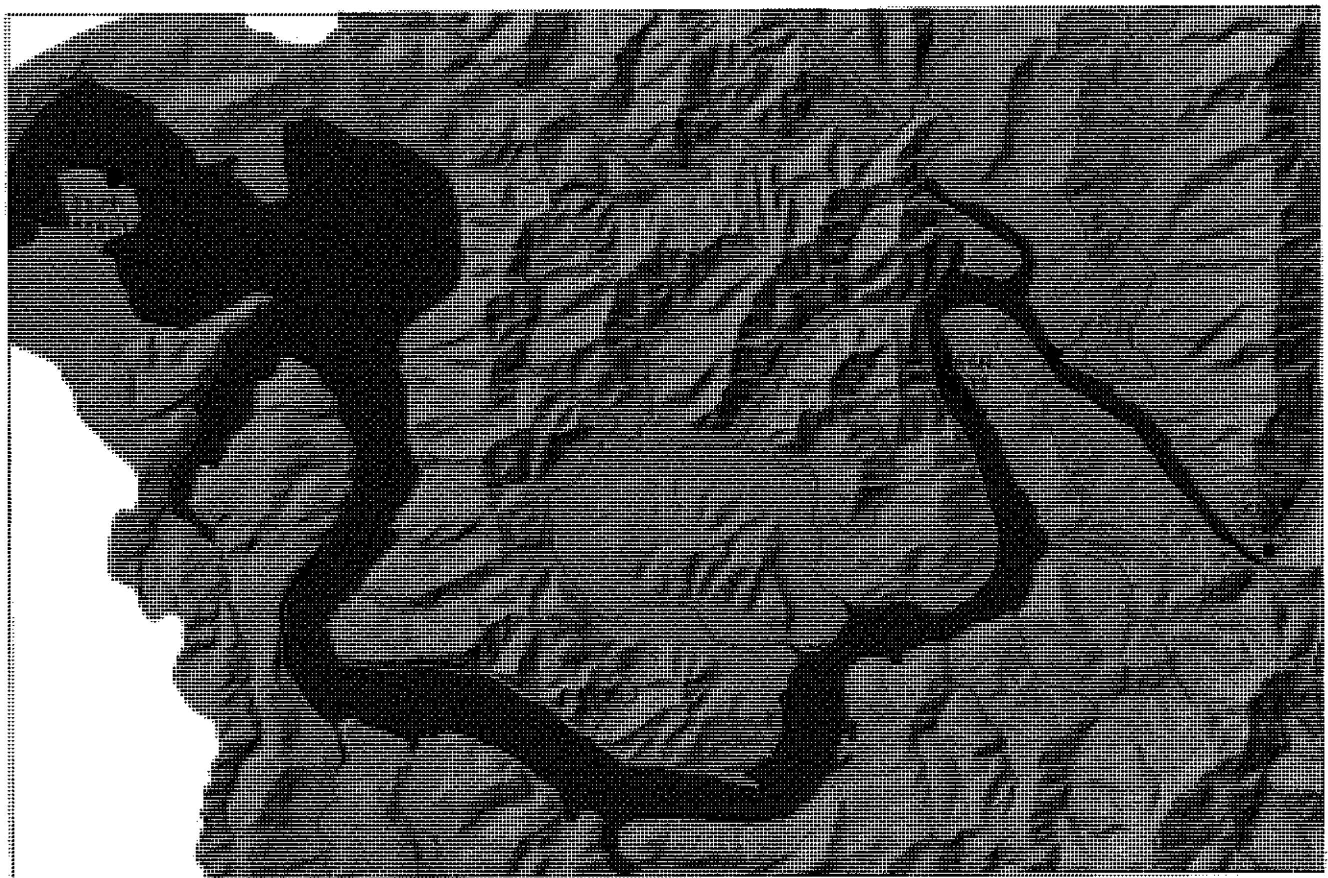

**Figure 15: Water temperature monitoring sites in Yaquina Bay and River**

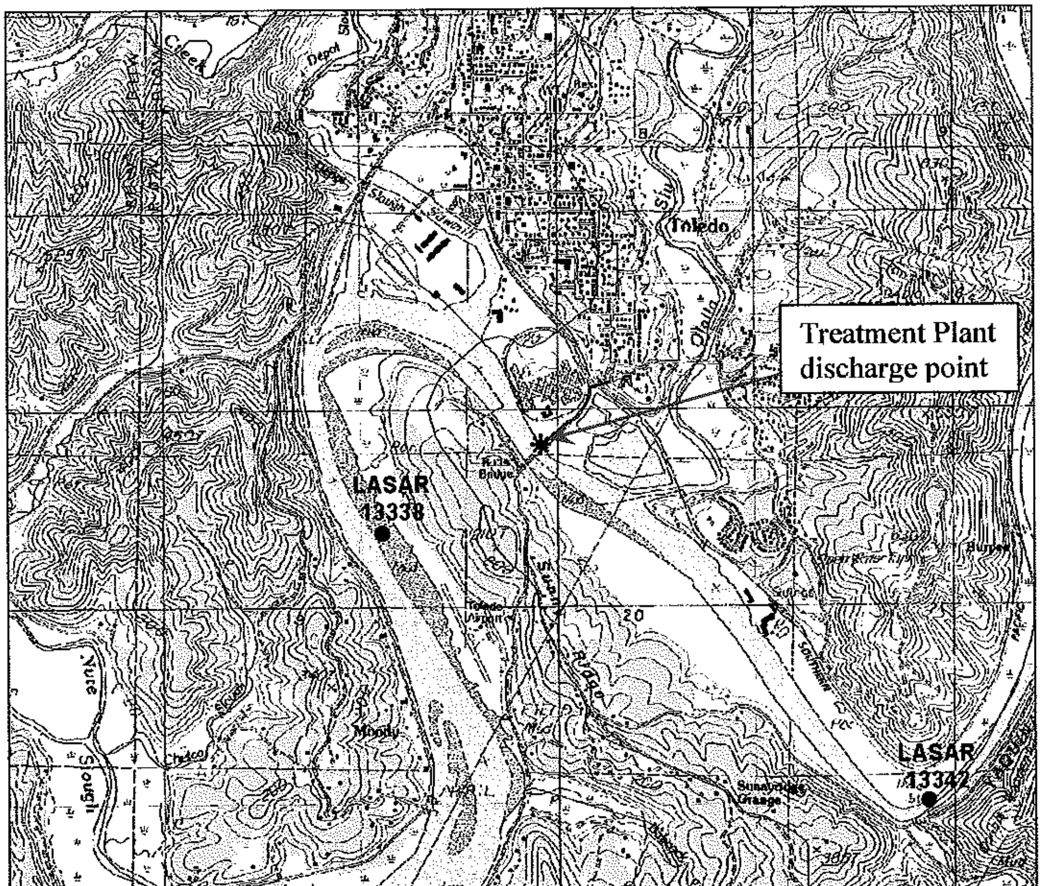


**Figure 19: Oregon Department of Environmental Quality water temperature monitoring sites with most data and closest to City of Toledo WWTP discharge**

**Table 5: Water temperature monitoring sites (with most data) and extent of data for Yaquina Bay and River**

| Site ID | Site Description | Years of data | Comment |
|---|---|---|---|
| LASAR 13338 | Yaquina River at Old Shingle Mill Ramp (ODEQ) | 1960 to 2001 | Downstream of discharge point |
| LASAR 13342 | Yaquina River at Mill Creek (Toledo) (ODEQ) | 1960 to 2001 | Upstream of discharge point |
| 9435380 | South Beach, Yaquina River (NOAA-OSU) | 1991 to 2005 | Current, Active gage |

**Table 6: Water temperature data counts for each month**

| Month | South Beach Temperature Data Count | LASAR 13342 Temperature Data Count | LASAR 13338 Temperature Data Count |
|---|---|---|---|
| January | 10,317 | 9 | 10 |
| February | 9,381 | 6 | 8 |
| March | 10,284 | 15 | 18 |
| April | 9,960 | 24 | 27 |
| May | 9,399 | 16 | 15 |
| June | 8,676 | 10 | 11 |
| July | 9,495 | 10 | 12 |
| August | 9,483 | 10 | 11 |
| September | 9,086 | 14 | 14 |
| October | 9,412 | 14 | 12 |
| November | 9,189 | 14 | 14 |
| December | 9,522 | 7 | 7 |

The data from each site were separated into groups by month (over the date range of data) and ranked within each month by lowest to highest values and given a probability of occurrence based on the total number for each month. Since the South Beach site has thousands of values per month the frequency curves were expected to be smoother than the curves from site with much less data.

Figure 20 through Figure 25 show water temperature frequency plots for each month of the year for the three monitoring sites. The figures show in the winter, November through April, that the two monitoring sites near the City of Toledo outfall have cooler temperatures than the bay. This may be due to higher river flows which are colder than the bay and which may dominate in this section of the river/bay. In the summer, May to October, when river flows are lower the temperatures at these two are higher than the bay. This may be due to lower river flows and the section of the river being dominated more by tidal flushing than upstream river flow. The frequency curves also indicate there is not much difference in temperatures between the two monitoring sites near the city's outfall. The frequency curves also indicate there is a higher probability of this whole reach of river having higher temperatures than downstream in the bay.

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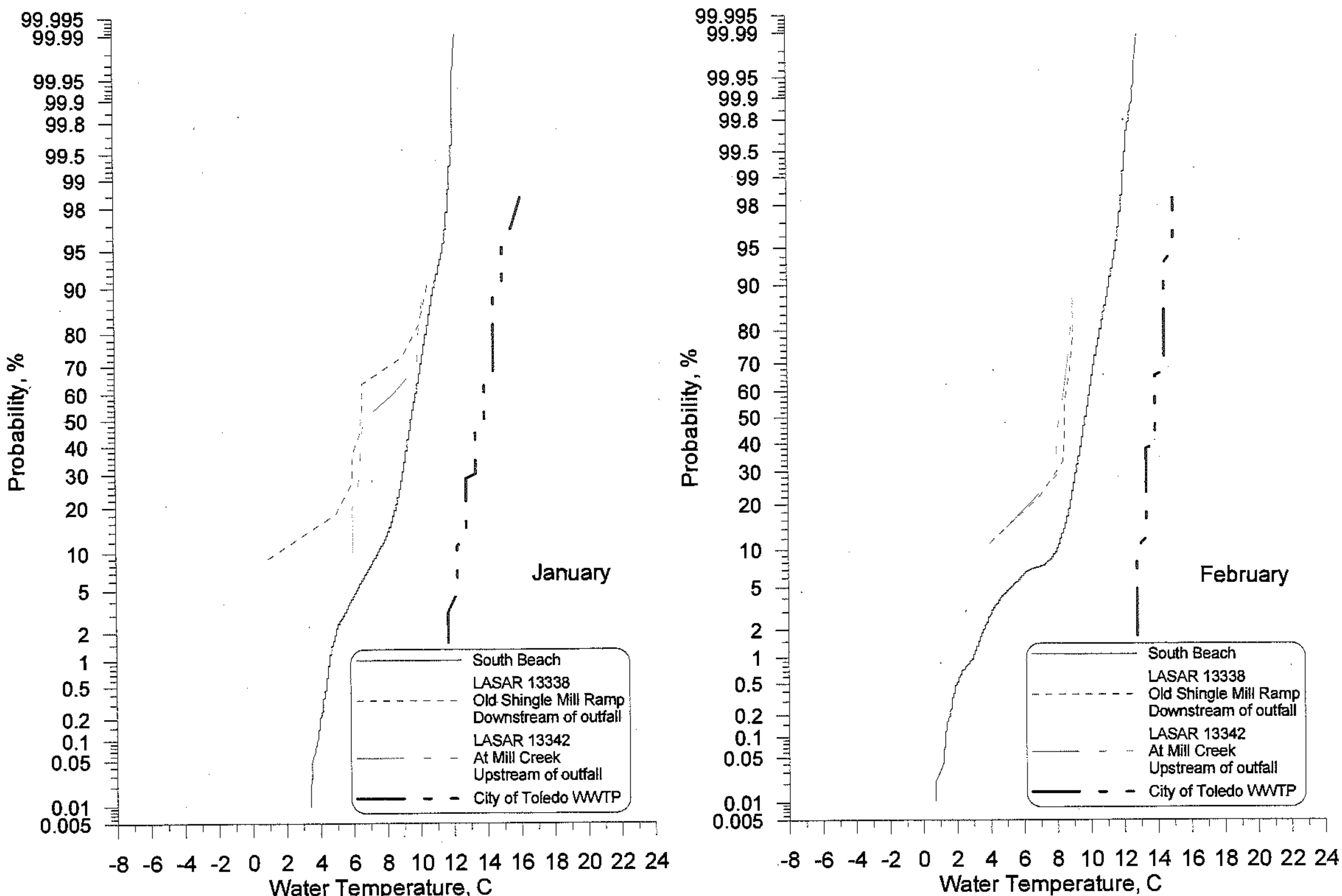


**Figure 20: Water temperature frequency in Yaquina Bay and Yaquina River and the City of Toledo WWTP effluent for January and February**

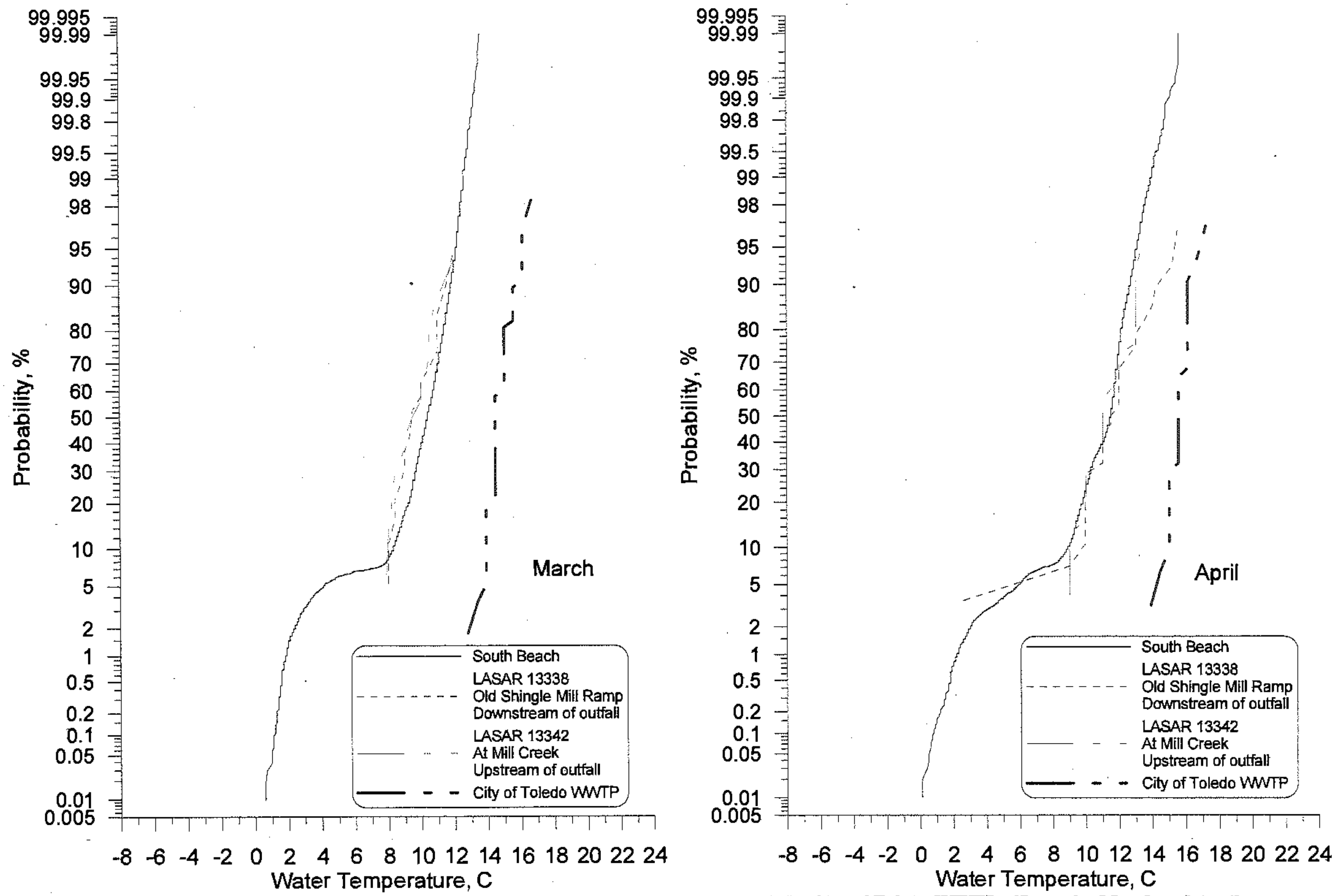


**Figure 21: Water temperature frequency in Yaquina Bay and Yaquina River and the City of Toledo WWTP effluent for March and April**

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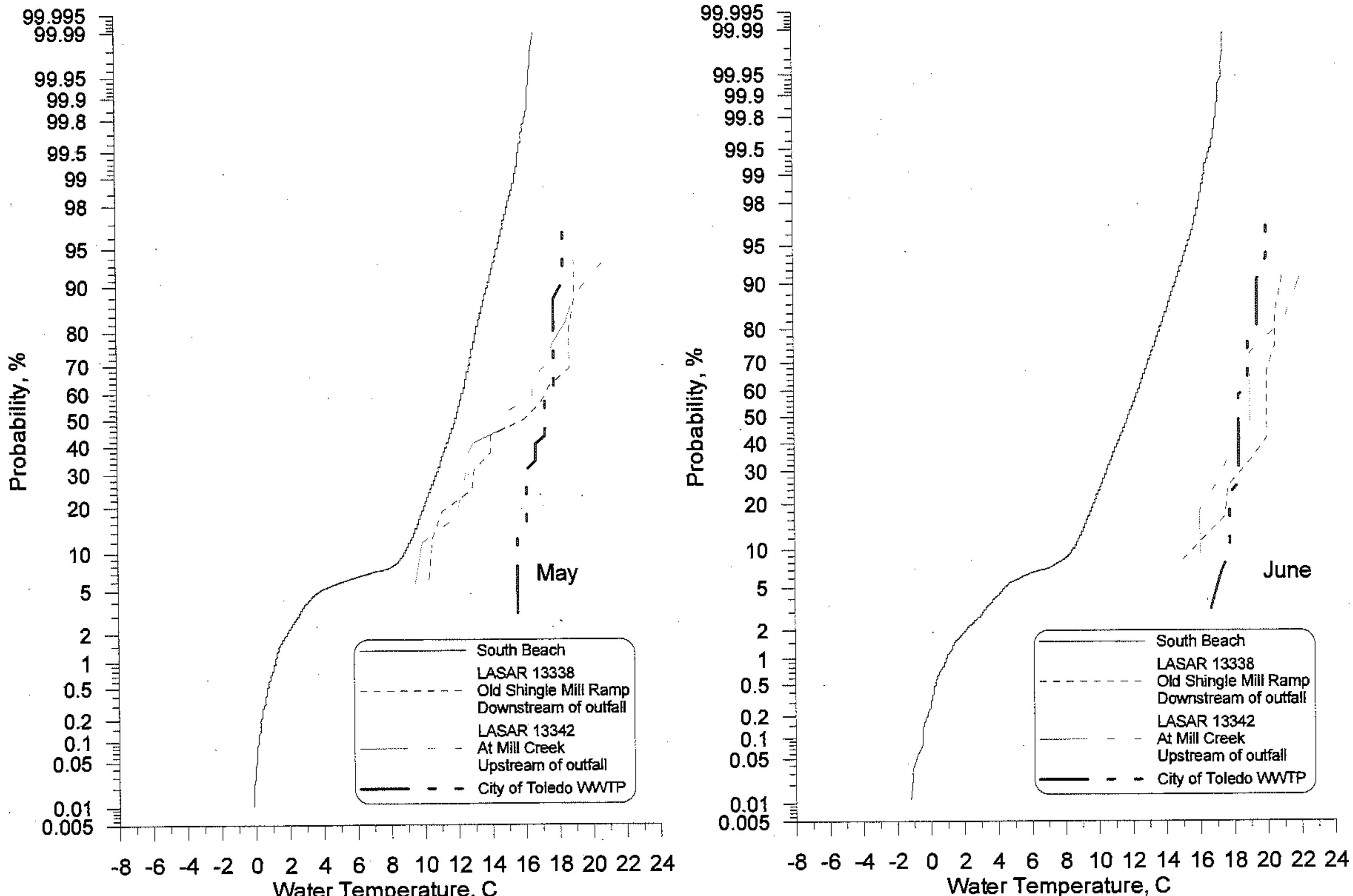


**Figure 22: Water temperature frequency in Yaquina Bay and Yaquina River and the City of Toledo WWTP effluent for May and June**

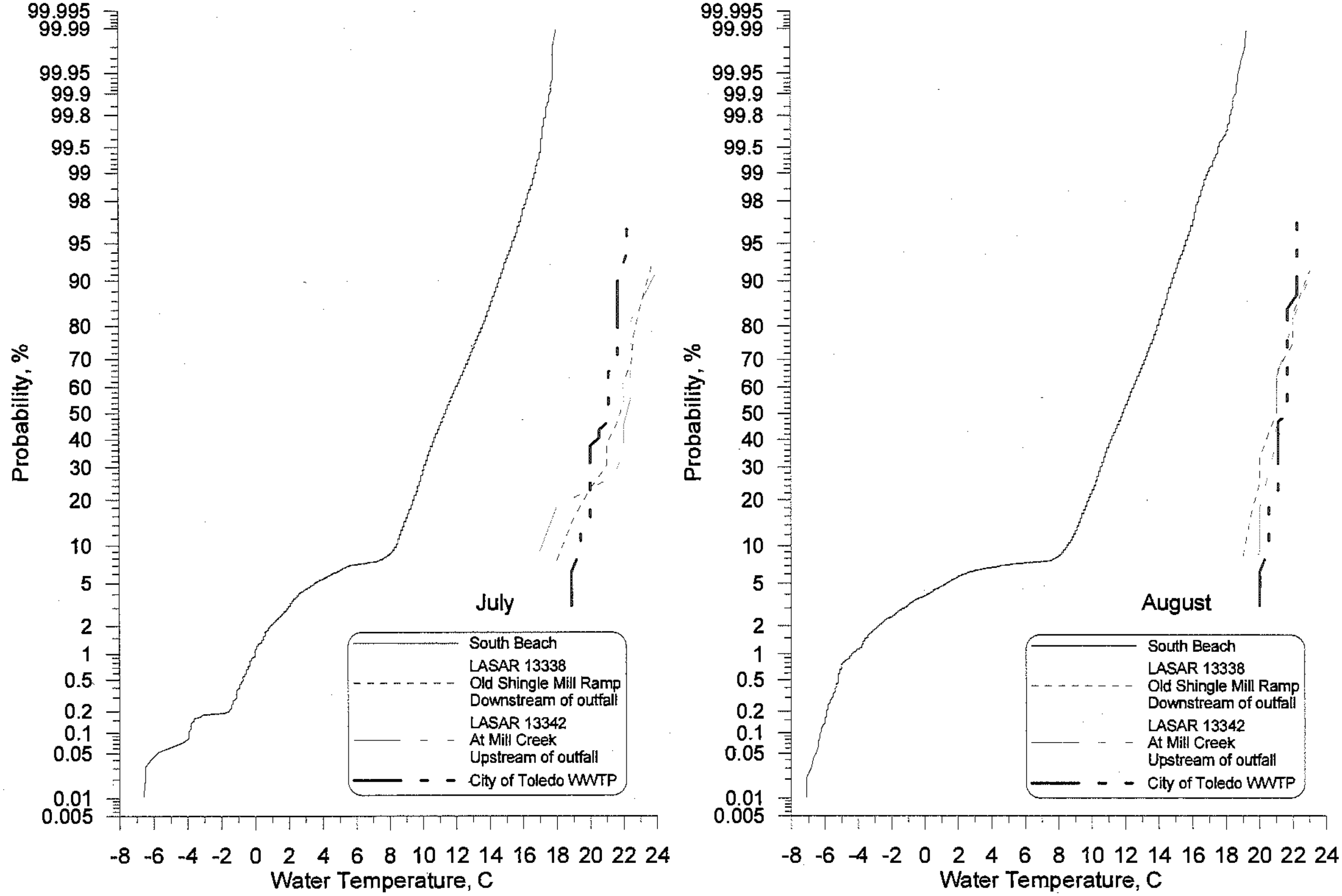


**Figure 23: Water temperature frequency in Yaquina Bay and Yaquina River and the City of Toledo WWTP effluent for July and August**

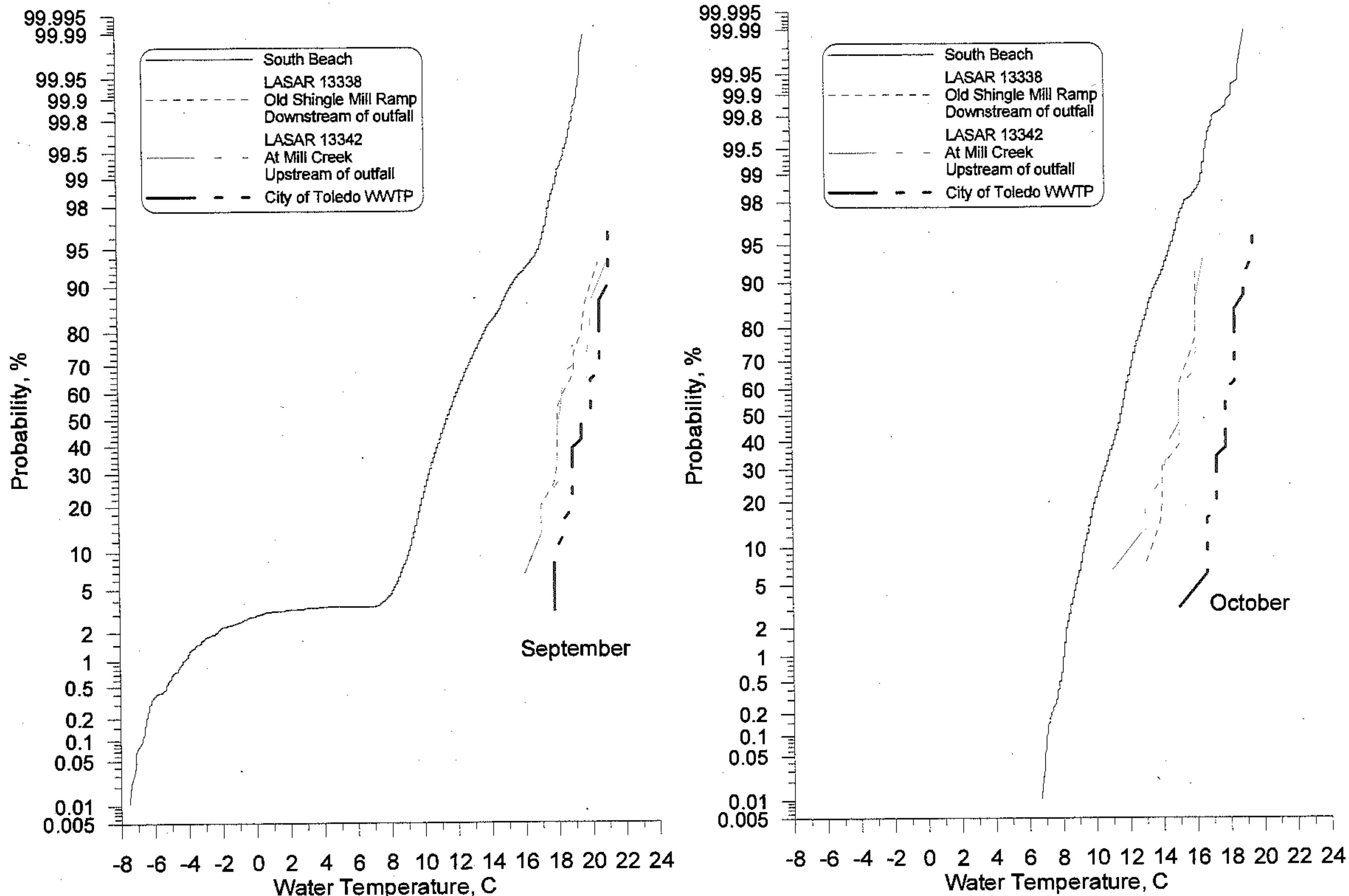


Figure 24: Water temperature frequency in Yaquina Bay and Yaquina River and the City of Toledo WWTP effluent for September and October

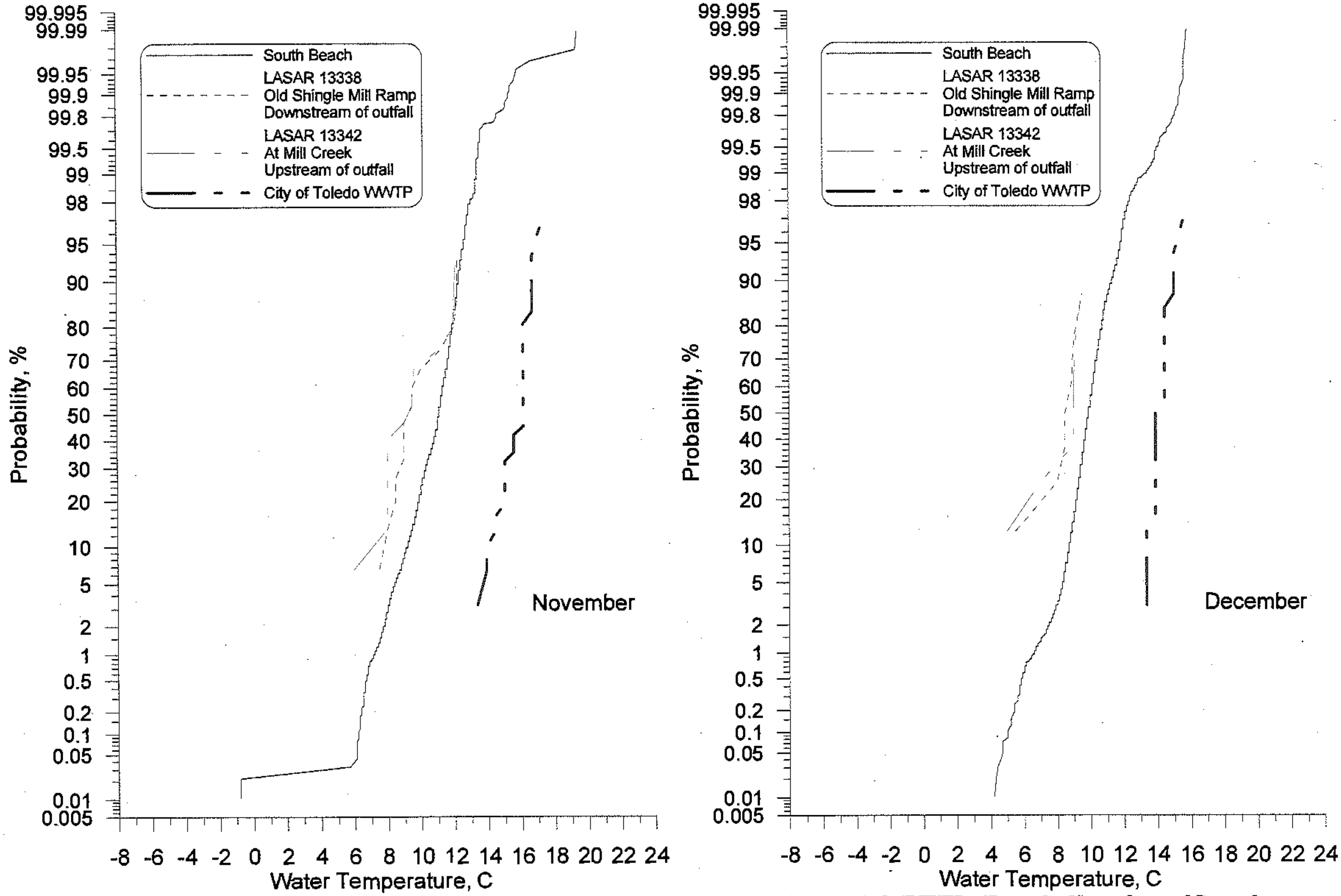


**Figure 25: Water temperature frequency in Yaquina Bay and Yaquina River and the City of Toledo WWTP effluent for November and December**

## *Yaquina River Flow Analysis*

There is limited flow data along the Yaquina River, but daily flow data was obtained from the USGS gage station at Chitwood (14306030) from between 1972 and 1991 (6,939 points). Figure 26 shows a map of Yaquina Bay and River up to the USGS gage station at Chitwood. The flows at the gage station were adjusted to account for the drainage basin area between Chitwood and Butler Bridge by multiplying the flow by 1.8 based on work by Furfari (1985).

The adjusted daily average flows at Butler Bridge were then used to calculate the 7Q10 and 1Q10 flows at the bridge for each month of the year. The 7Q10 flow is defined as the seven-day (weekly) low flow over a period of 10 years. The 1Q10 flow is the daily low flow over a period of 10 years. The 7Q10 is a typical low flow value used by state regulators to evaluate water quality compliance. Table 7 lists the monthly 7Q10 and 1Q10 flows in $ft^3/s$ and $m^3/s$. Figure 27 shows a bar chart of the 7Q10 flows at Butler Bridge and Figure 28 shows the 1Q10 flows at Butler Bridge over the year. The table and figures indicate the lowest 7Q10 flow occurs in September with a flow 0.24 $m^3/s$. This corresponds to the seasonal dry period, late in summer, before winter rains return and increase river flows.

Figure 29 shows a frequency of occurrence curve for the daily flows at Butler Bridge based on the adjusted data from the USGS gage at Chitwood. Figure 30 shows flow frequency curves for August, September and October separated by month. The figure indicates the flows in August and September are lower than in October as expected in the late summer dry period, whereas October sees increases in flow due to fall rain events. Table 7: Yaquina River at Butler Bridge 7Q10 and 1Q10 flows

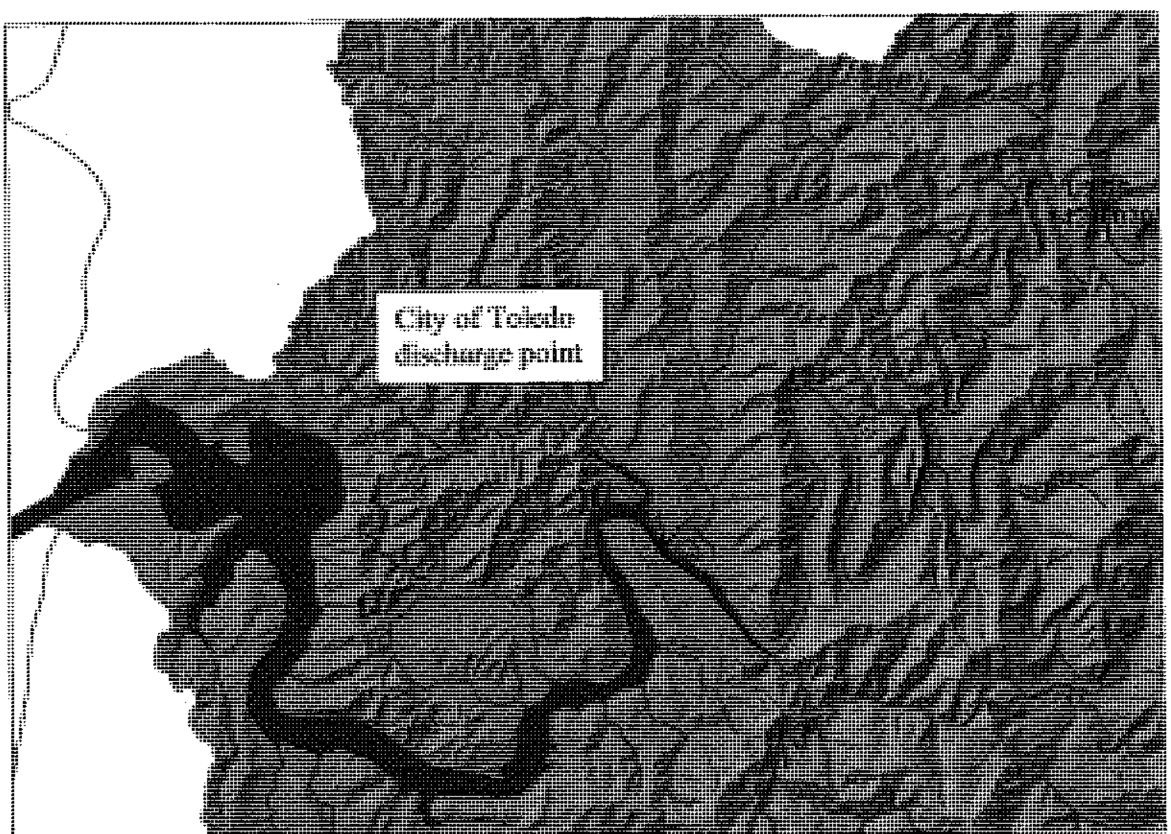


**Figure 26: U.S. Geological Survey gage station on the Yaquina River near Chitwood (14306030)**

**Table 7: Yaquina River at Butler Bridge 7Q10 and 1Q10 flows**

| Month | 1Q10 at Toledo, cfs | 1Q10 at Toledo, $m^3/s$ | 7Q10 at Toledo, cfs | 7Q10 at Toledo, $m^3/s$ |
|---|---|---|---|---|
| Jan | 122.0 | 3.45 | 128.6 | 3.64 |
| Feb | 95.1 | 2.69 | 106.3 | 3.01 |
| Mar | 172.8 | 4.89 | 184.8 | 5.23 |
| Apr | 120.0 | 3.40 | 128.7 | 3.64 |
| May | 75.5 | 2.14 | 81.1 | 2.30 |
| Jun | 38.8 | 1.10 | 41.0 | 1.16 |
| Jul | 17.0 | 0.48 | 17.6 | 0.50 |
| Aug | 9.9 | 0.28 | 10.7 | 0.30 |
| Sep | 7.4 | 0.21 | 8.6 | 0.24 |
| Oct | 8.9 | 0.25 | 9.8 | 0.28 |
| Nov | 19.1 | 0.54 | 25.4 | 0.72 |
| Dec | 77.1 | 2.18 | 86.4 | 2.45 |

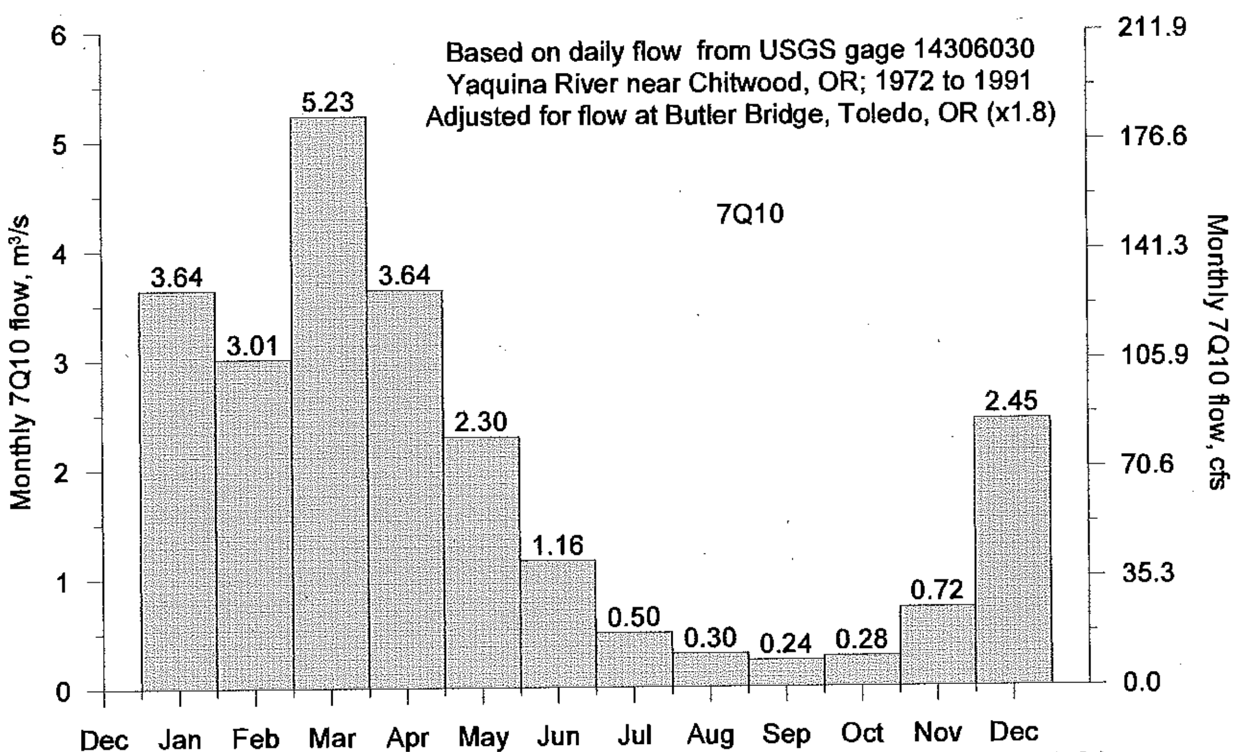


**Figure 27: Monthly 7Q10 low flow on the Yaquina River at Butler Bridge (City of Toledo)**

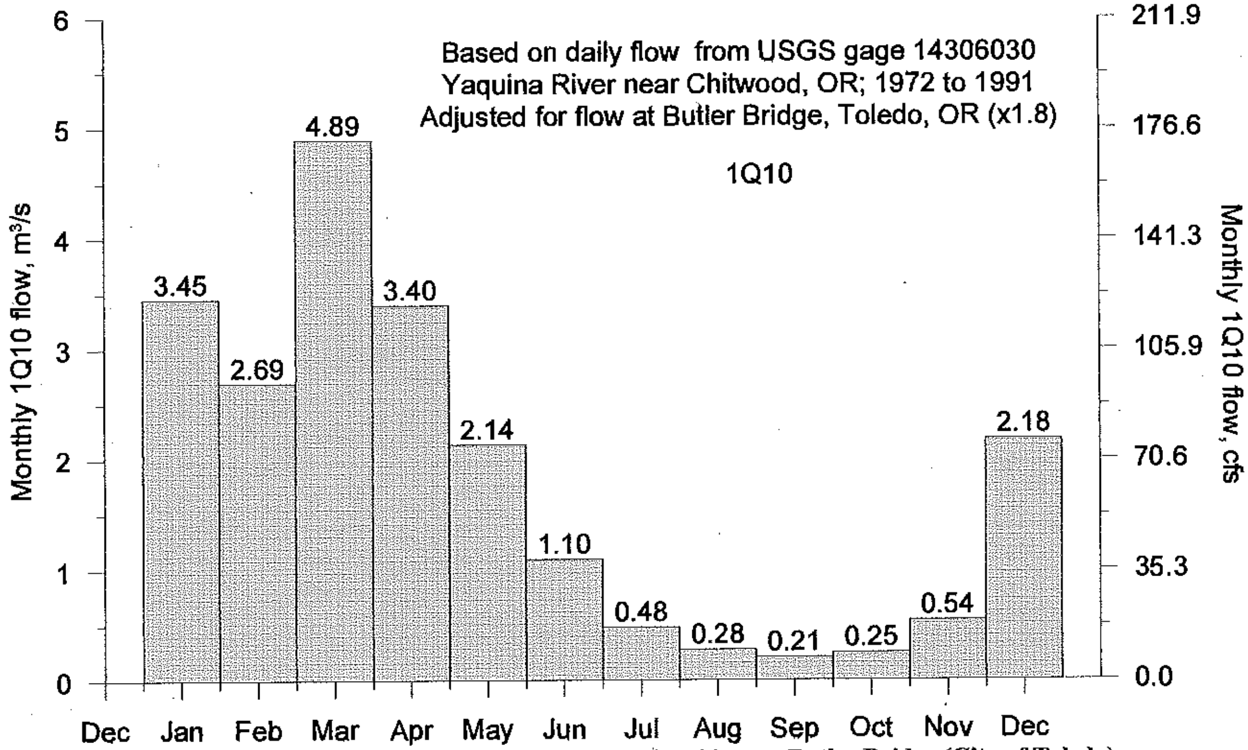


**Figure 28: Monthly 1Q10 low flow on the Yaquina River at Butler Bridge (City of Toledo)**

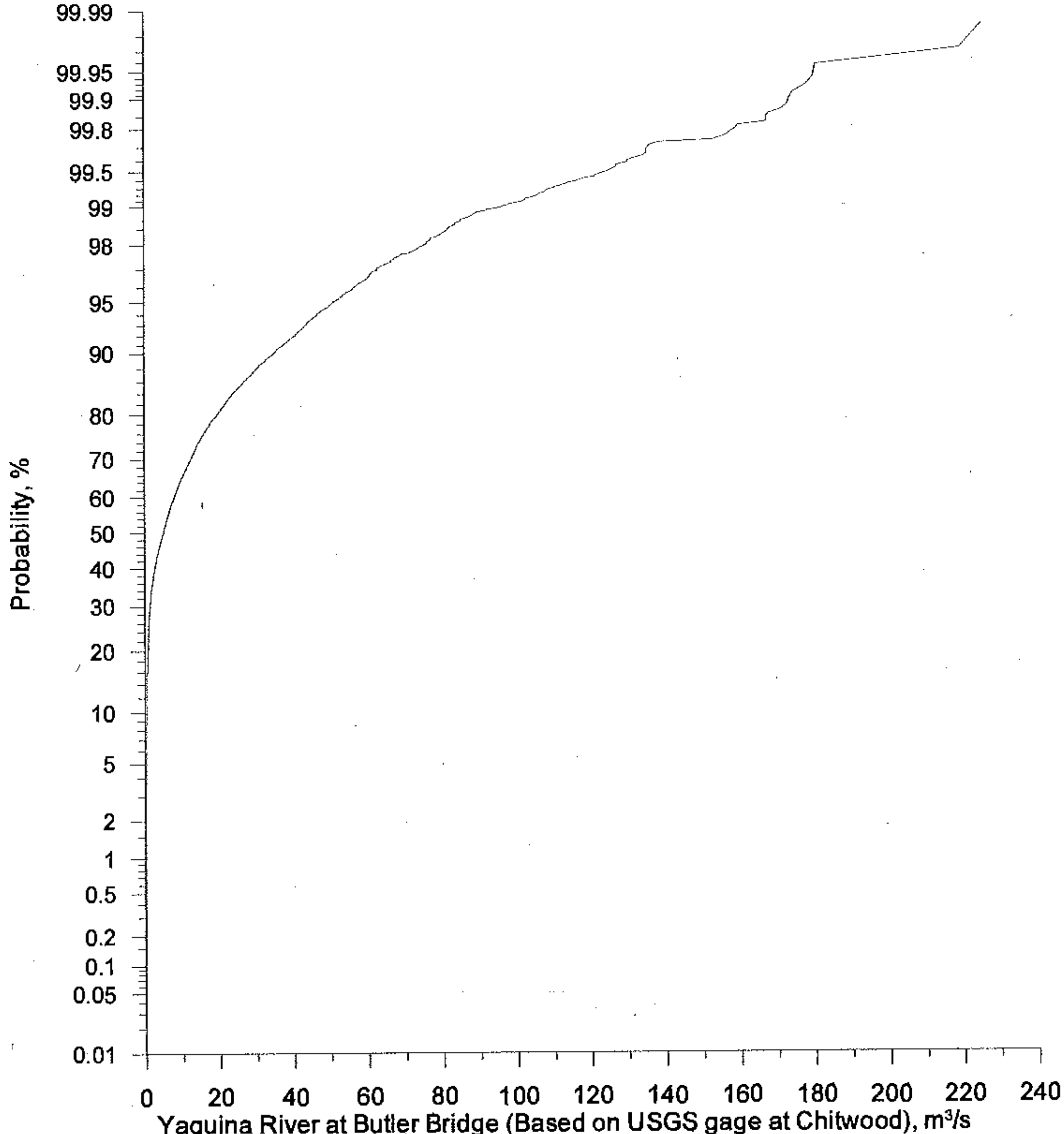


**Figure 29: Yaquina River at Butler Bridge flow frequency, data from 1972 to 1991**

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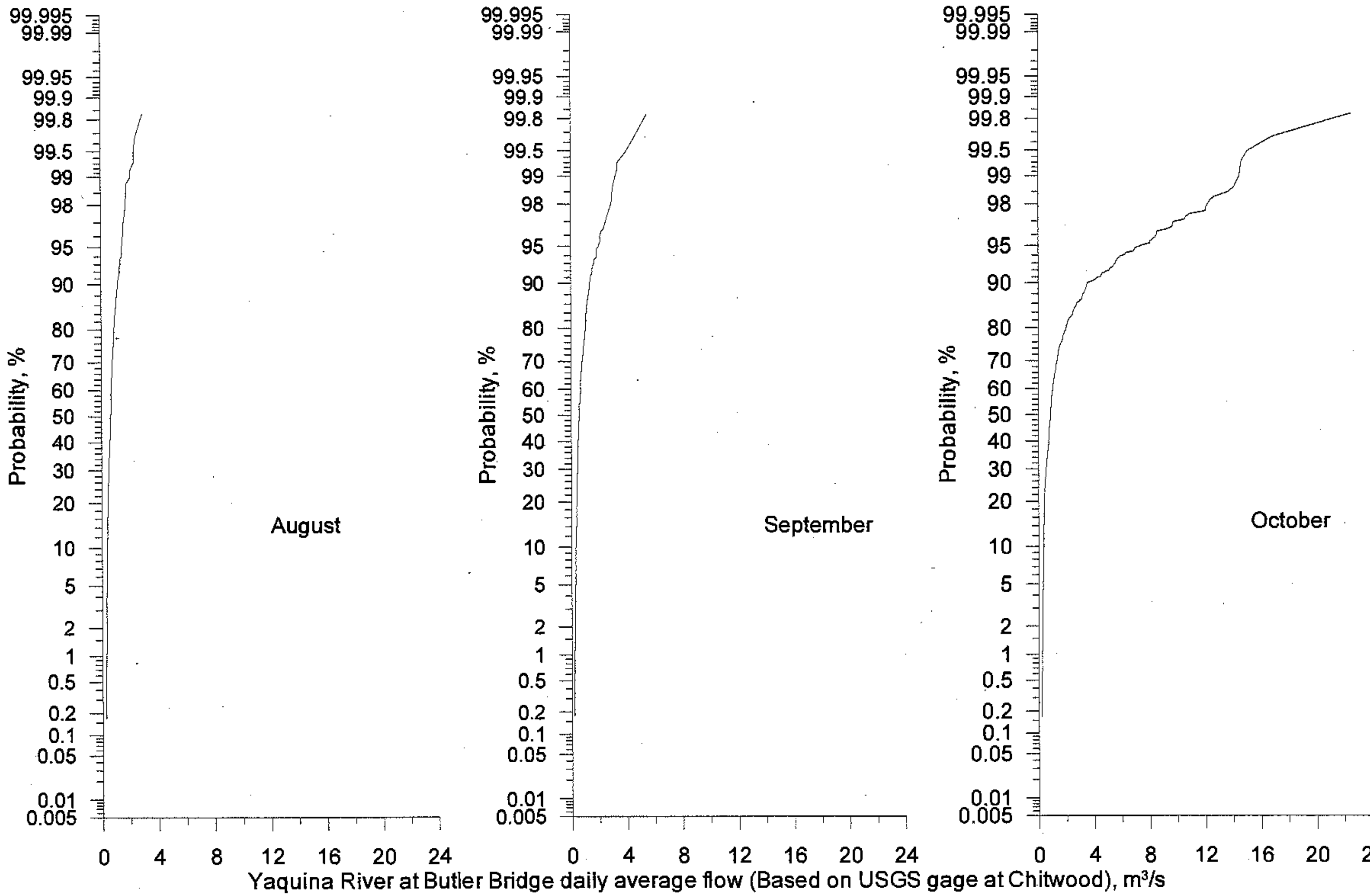


**Figure 30: Yaquina River at Butler Bridge flow frequency for August to October, data from 1972 to 1991**

## *River Morphology*

The Yaquina River morphology near the City of Toledo outfall was developed from several pieces of data. The U.S. Army Corps of Engineers (ACOE) conducted hydrographic surveys of Yaquina Bay and River up to Butler Bridge in 2000, 2004 and most recently in 2005. Figure 31 shows a section of the Yaquina River in GIS with location of the outfall and the location of hydrographic survey points in the river. In addition to the survey data topographic data was obtained for the river channel banks from the U.S. Geological Survey's DEM (Digital Elevation Model) of this reach of river. Figure 31 shows the river bank elevation points. The ACOE hydrographic survey data were provided as water depths relative to MLLW in feet. The survey data was converted to elevation relative to NGVD29 datum using the relationship:

$$NGVD29, m = MLLW, m - 1.2172 \ m$$

It should be noted this datum conversion is slightly different than the water level datum conversion used by NOAA and in the water level frequency analysis above. The differences in the datum conversion are very small and should have negligible impact on the results. Different datum conversions were used to be consistent with the data sources and to put all analyses into a datum of NGVD29.

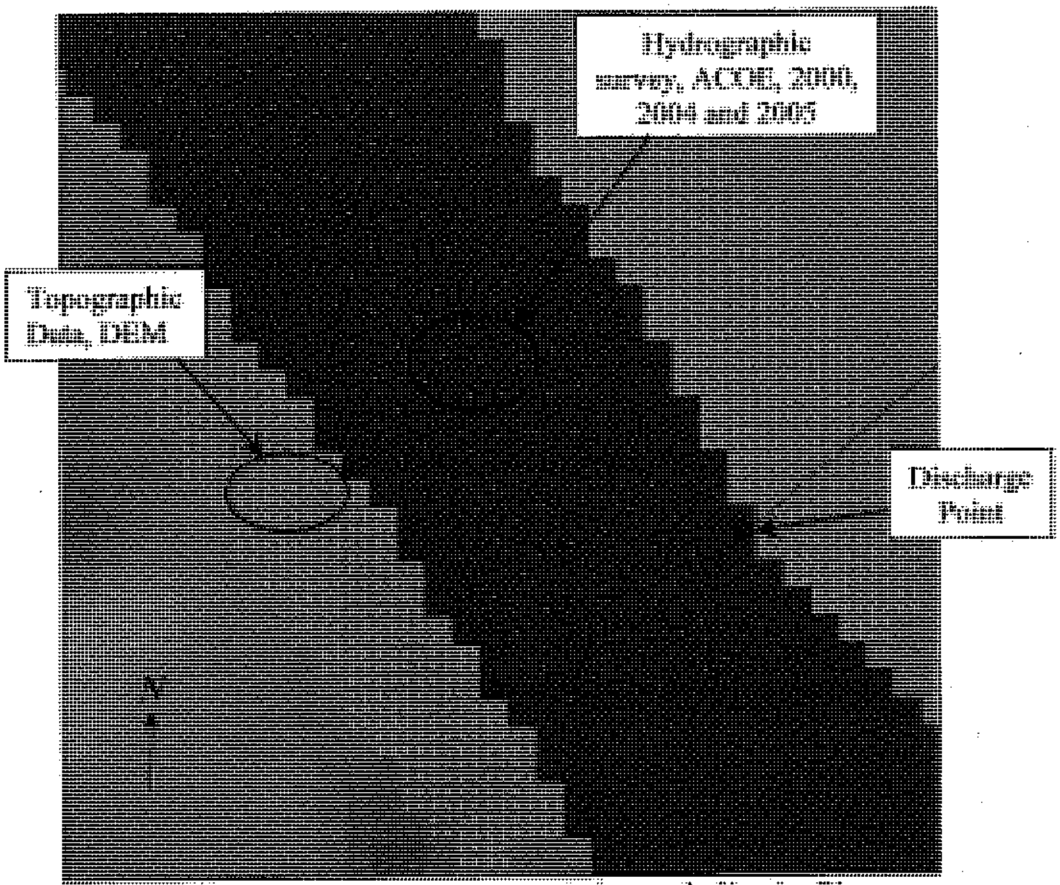


Figure 31: Hydrographic and topographic data on the Yaquina River

Figure 31 shows there are several data gaps between the ACOE hydrographic survey points and the topography data obtained from the DEM. In order to develop a more representative river cross section additional points were digitized in a geographic information system database as shown in Figure 32. The elevations associated with these points were determined by linearly interpolating between the DEM data on the river banks and the nearest hydrographic survey points.

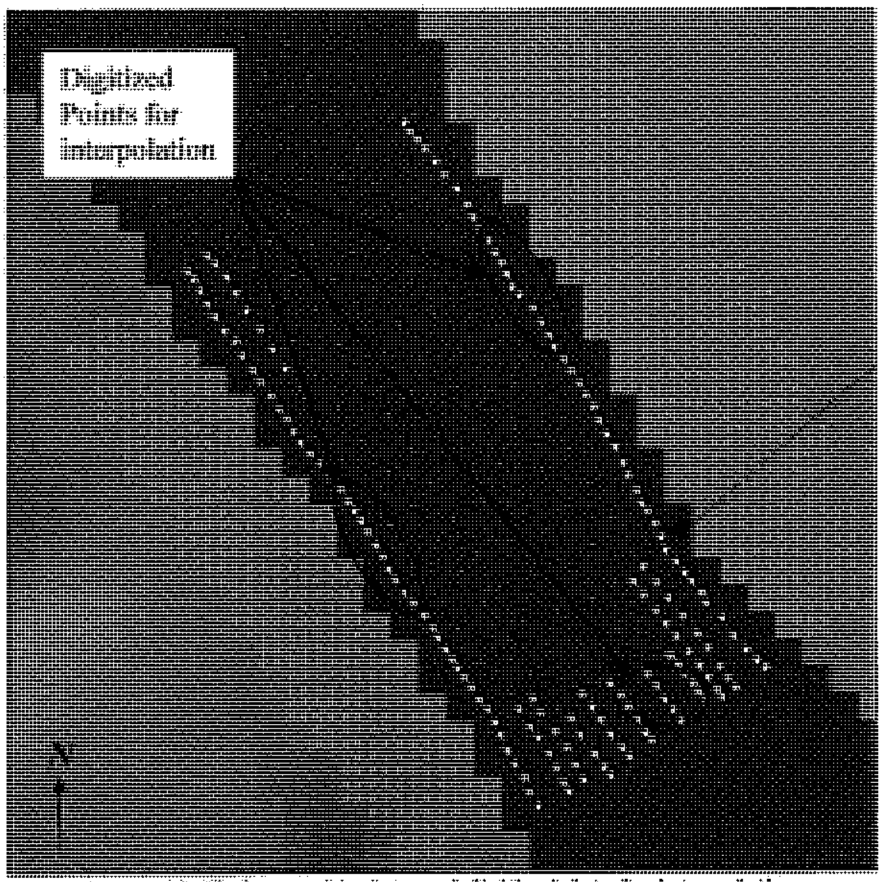


Figure 32: Hydrographic data and digitized data for interpolation

The digitized points were then combined with bank elevation data and the ACOE hydrographic survey data in the region around the outfall and used in a contour plotting software, SURFER, to develop an elevation contour plot of the river morphology. Figure 33 shows a contour plot of the river bottom elevation at the outfall location with the location of the outfall and the river cross section at the outfall.

Figure 34 shows a side view surface plot of the river morphology looking down stream and Figure 35 shows a perspective view surface plot of the river channel looking down stream. All three figures indicate there are deeper locations in the river channel cross section at the outfall location. A cross section of the river channel is shown in Figure 36 with the location of the outfall structure and pipe. As the cross section indicates there is a deeper area in the river cross section and the side slopes are high.

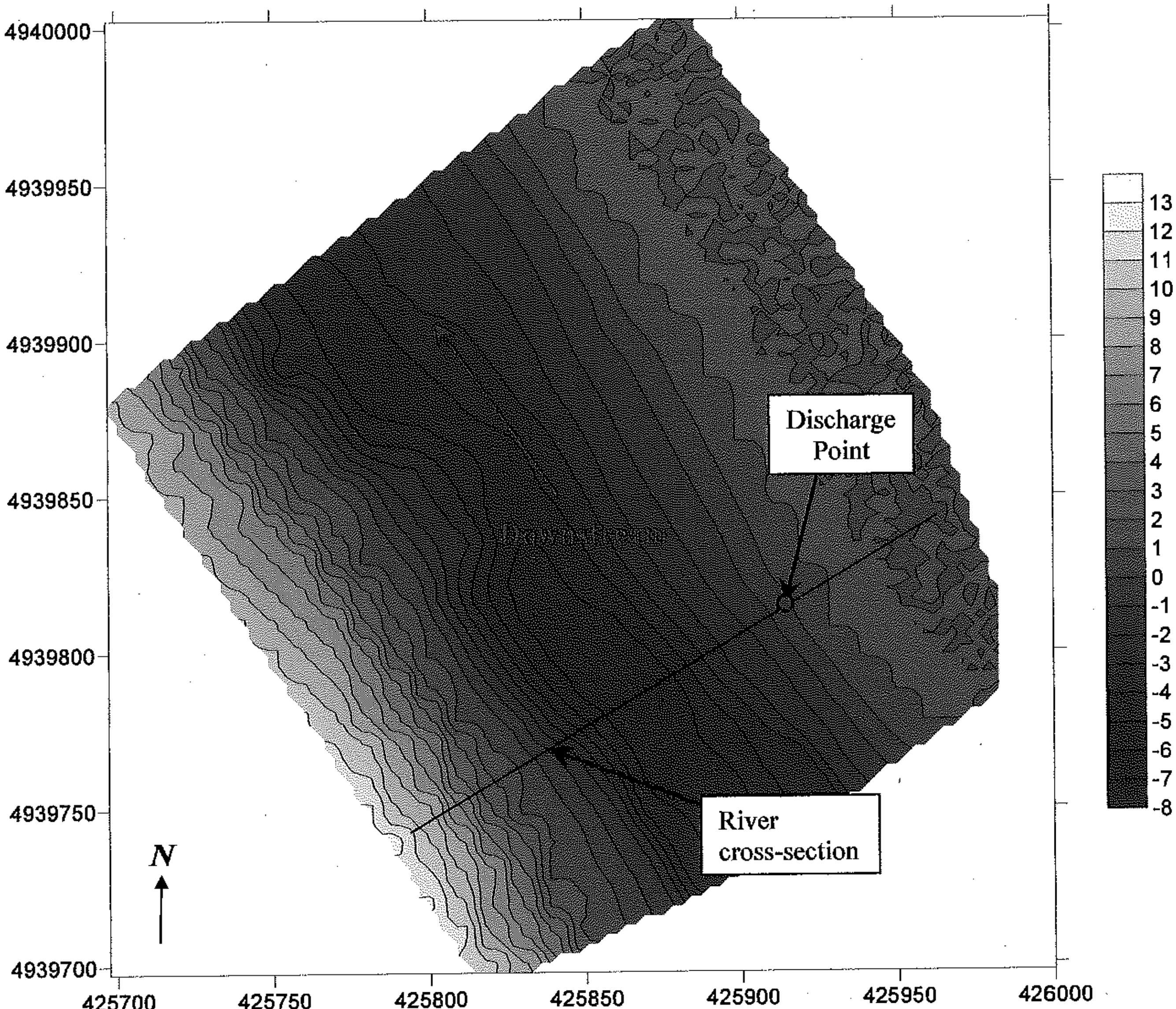


**Figure 33: Yaquina River elevation contour surround the City of Toledo WWTP discharge point**

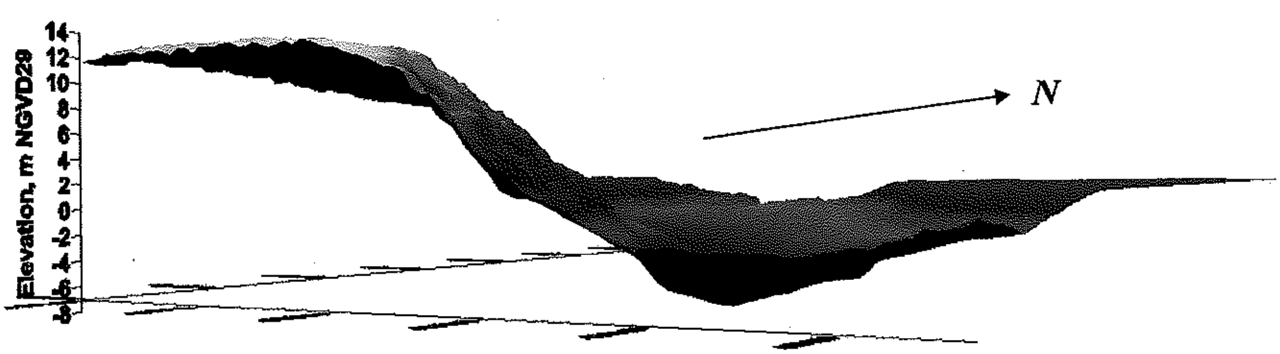


**Figure 34: Side view looking downstream ou the Yaquina River**

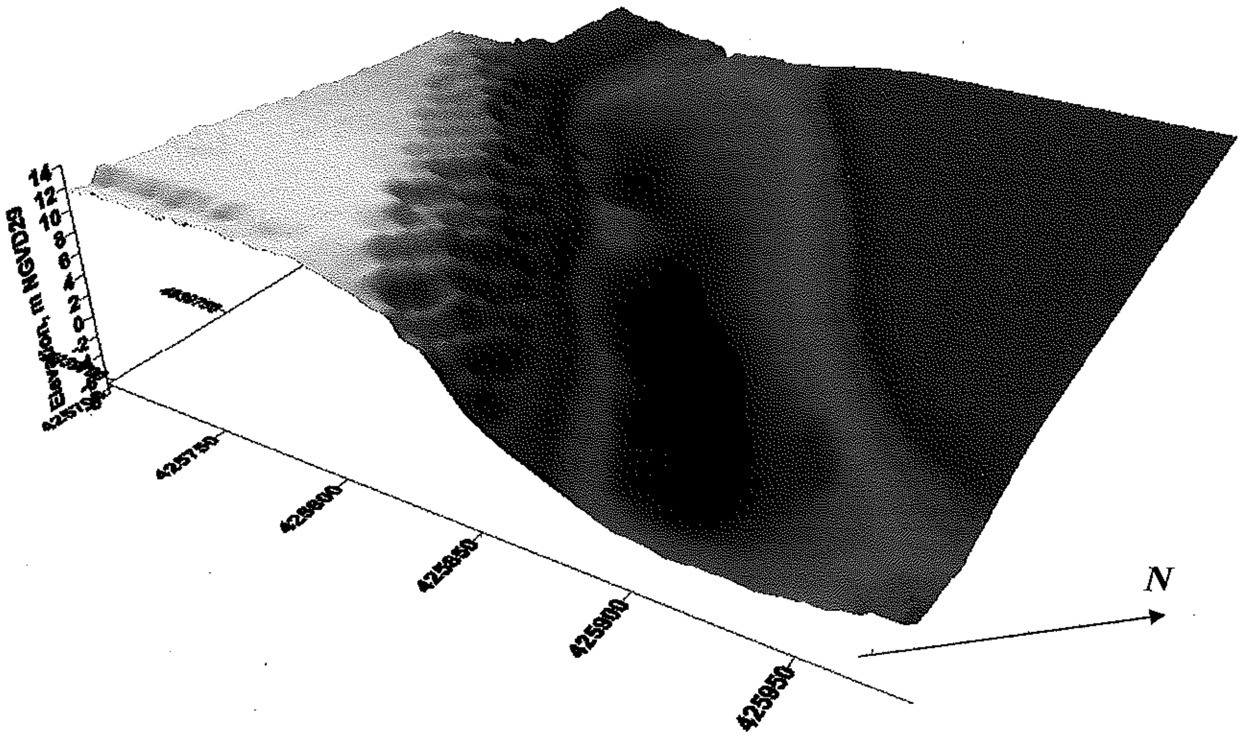


**Figure 35: Perspective view looking downstream on the Yaquina River**

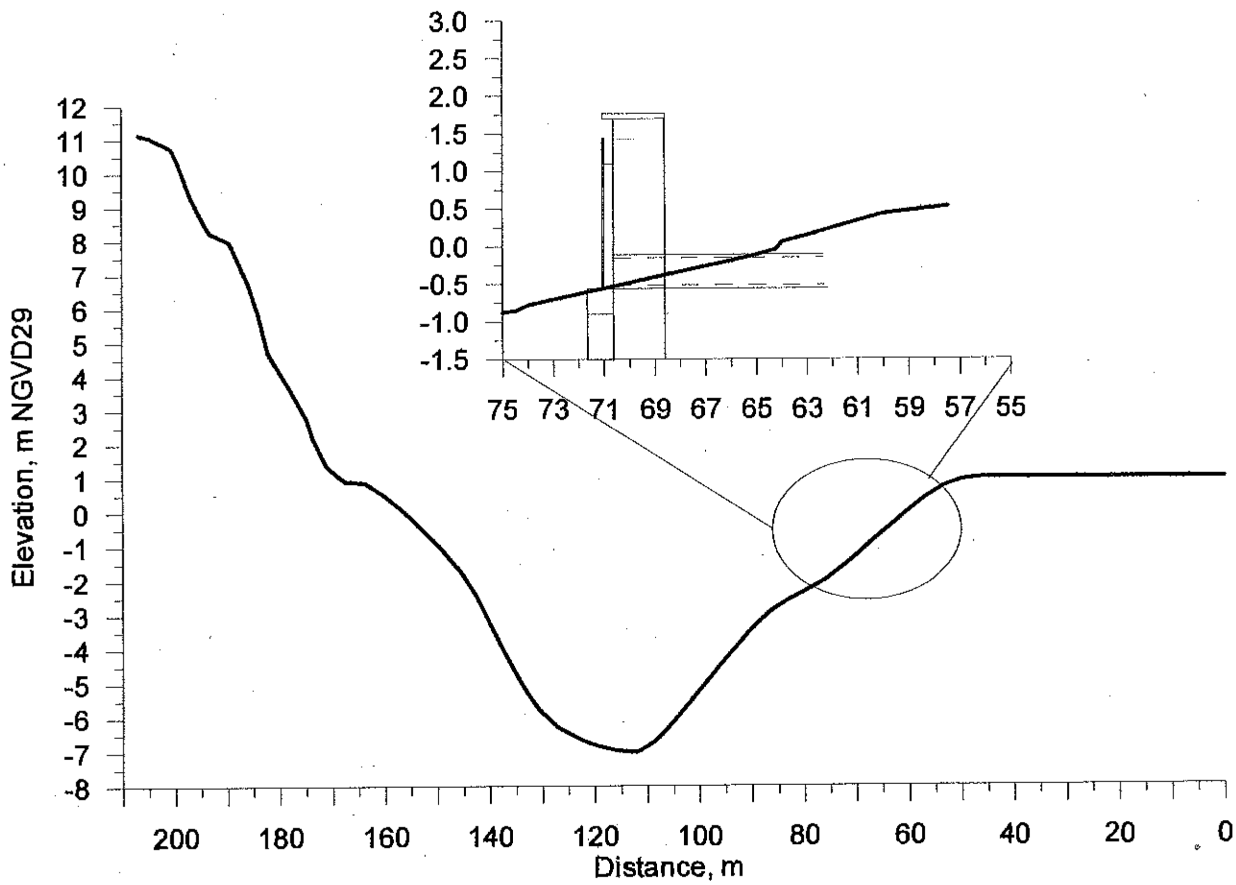


**Figure 36: Bathymetric cross-section of the Yaquina River at the discharge point**

## *Resource Maps*

In order to better understand the uses along Yaquina Bay and River several resource maps were generated to detail historical water quality monitoring sites, National Pollutant Discharge Elimination System (NPDES) Permitting Program sites, shellfish areas and beach and water access points.

### Historical Water Quality Monitoring sites

There have been primarily three agencies monitoring the water quality and sediment quality in Yaquina Bay and River: the Oregon Department of Environmental Quality, the Oregon Department of Human Services, and the Oregon Department of Agriculture. Figure 37 shows a map of Yaquina Bay and the monitoring sites of the agencies between 1960 and 2004. Table 8 lists the monitoring site locations shown Figure 37, the corresponding agency and the date range of data. More recent data was not available because the data was not in the ODEQ LASAR system (conversation with ODEQ staff).

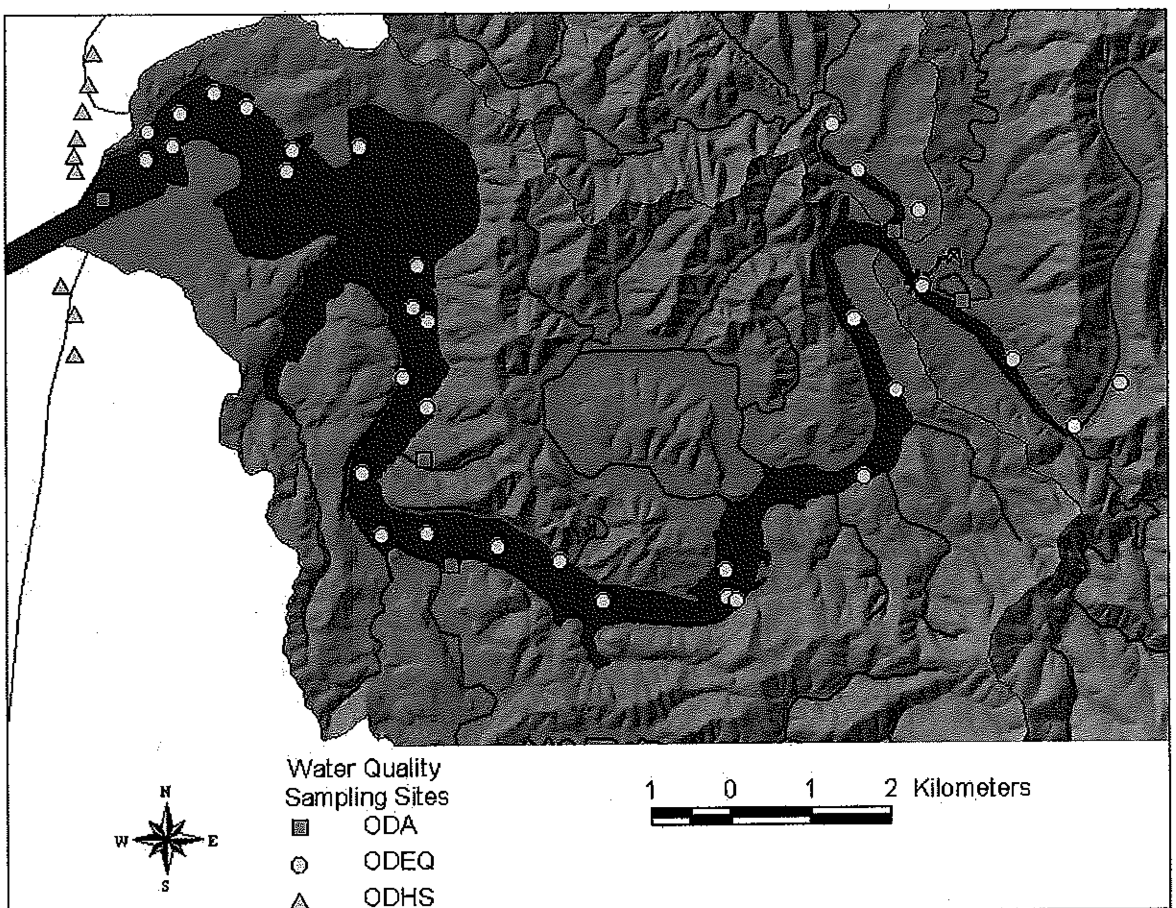


**Figure 37: Historical water quality monitoring sites in Yaquina Bay and River**

**Table 8: Historical water quality monitoring site locations in Yaquina Bay and River**

| Site ID | Site Description | UTMY, m | UTMX, m | Agency | Data Type | Minimum Date | Maximum Date |
|---|---|---|---|---|---|---|---|
| 10583 | Yaquina River D/S Toledo | 4938557.69 | 428445.77 | ODEQ | LASAR | 06/21/1966 | 06/13/1967 |
| 12329 | Gaper Station 11A (Yaquina Bay @ Marker #7) | 4940877.16 | 415736.05 | ODA | LASAR | 01/10/2000 | 08/19/2002 |
| 13322 | Yaquina Bay 600 Yds South Of Marker #17 | 4939527.91 | 419615.37 | ODEQ | LASAR | 03/14/1960 | 12/10/1968 |
| 13323 | Yaquina Bay 100 Yds South Of Marker #17 | 4939370.13 | 419795.96 | ODEQ | LASAR | 03/14/1960 | 07/01/1969 |
| 13324 | Yaquina Bay at Marker #19 (Weiser Point) | 4938292.65 | 419782.62 | ODEQ | LASAR | 03/14/1960 | 08/19/2002 |
| 13325 | Yaquina Bay at Oneatta Point (Near Marker #21) | 4937469.44 | 418978.53 | ODEQ | LASAR | 03/14/1960 | 04/30/1972 |
| 13326 | Yaquina River at Marker #25 | 4936715.12 | 419778.98 | ODEQ | LASAR | 03/14/1960 | 08/19/2002 |
| 13327 | Yaquina River at Marker #26 | 4936548.74 | 420658.28 | ODEQ | LASAR | 03/14/1960 | 08/19/2002 |
| 13328 | Yaquina River at Oregon Oyster | 4936361.53 | 421434.15 | ODEQ | LASAR | 03/14/1960 | 08/19/2002 |
| 13329 | Yaquina River at Marker #28 | 4935855.05 | 421975.94 | ODEQ | LASAR | 03/14/1960 | 08/19/2002 |
| 13330 | Yaquina River at Marker #32 | 4935903.23 | 423524.98 | ODEQ | LASAR | 03/14/1960 | 08/19/2002 |
| 13331 | Yaquina River at Marker #34 | 4936236.75 | 423505.10 | ODEQ | LASAR | 03/14/1960 | 09/22/1970 |
| 13332 | Yaquina Bay at Hwy 101 (Yaquina Bay Bridge) | 4941392.13 | 416290.22 | ODEQ | LASAR | 08/18/1960 | 08/19/2002 |
| 13333 | Yaquina Bay at Mclean Point | 4941490.84 | 418124.30 | ODEQ | LASAR | 08/18/1960 | 08/19/2002 |
| 13334 | Yaquina River at Coquille Point | 4940049.39 | 419669.45 | ODEQ | LASAR | 08/18/1960 | 08/19/2002 |
| 13335 | Yaquina River at Marker #42 | 4937394.17 | 425233.57 | ODEQ | LASAR | 08/18/1960 | 07/01/1969 |
| 13336 | Yaquina River at Marker #47 | 4938467.08 | 425642.82 | ODEQ | LASAR | 08/18/1960 | 07/16/2002 |
| 13338 | Yaquina River at Old Shingle Mill Ramp | 4939372.87 | 425129.40 | ODEQ | LASAR | 08/18/1960 | 11/13/2001 |
| 13339 | Depot Slough at Mouth | 4940445.04 | 425602.04 | ODA | LASAR | 03/06/2001 | 11/13/2001 |
| 13340 | Yaquina River at Butler Street (Toledo) | 4939763.17 | 425967.20 | ODEQ | LASAR | 08/18/1960 | 04/19/1986 |
| 13341 | Yaquina River at Cascadia Mill | 4938839.34 | 427099.58 | ODEQ | LASAR | 08/18/1960 | 08/05/1980 |
| 13342 | Yaquina River at Mill Creek (Toledo) | 4938008.63 | 427868.17 | ODEQ | LASAR | 08/18/1960 | 11/13/2001 |
| 13343 | Ollala Slough at Mouth | 4939568.81 | 426449.09 | ODA | LASAR | 03/06/2001 | 11/13/2001 |
| 13345 | Pooles Slough at Mouth | 4936311.69 | 420059.85 | ODA | LASAR | 01/10/2000 | 08/19/2002 |
| 13349 | Yaquina River at Depot Slough | 4941216.53 | 425174.51 | ODEQ | LASAR | 01/31/1967 | 12/10/1968 |
| 13651 | Yaquina Bay South Beach Marina at Mouth | 4941543.33 | 416625.41 | ODEQ | LASAR | 01/18/1995 | 08/19/2002 |

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| Site ID | Site Description | UTMY, m | UTMX, m | Agency | Data Type | Minimum Date | Maximum Date |
|---|---|---|---|---|---|---|---|
| 13684 | Yaquina Bay @ Marker #20 | 4941535.88 | 418957.95 | ODEQ | LASAR | 03/09/1998 | 08/19/2002 |
| 13685 | Yaquina Bay @ Mccaferty`S Beds | 4936688.90 | 419206.98 | ODEQ | LASAR | 03/09/1998 | 08/19/2002 |
| 13686 | Parker Slough @ Mouth | 4937615.65 | 419726.61 | ODA | LASAR | 01/10/2000 | 08/19/2002 |
| 13690 | Yaquina Bay @ W. End Of Seawall | 4941964.52 | 416702.24 | ODEQ | LASAR | 07/28/1998 | 03/06/2002 |
| 13691 | Yaquina Bay @ Seawall @ Port Dock #7 | 4942214.40 | 417141.79 | ODEQ | LASAR | 07/28/1998 | 08/19/2002 |
| 13692 | Yaquina Bay @ E. End U/S Seawall | 4942031.59 | 417536.13 | ODEQ | LASAR | 07/28/1998 | 03/06/2002 |
| 13693 | Yaquina Bay @ Coast Guard Dock | 4941736.48 | 416294.68 | ODEQ | LASAR | 07/28/1998 | 03/06/2002 |
| 25644 | Yaquina Bay off North jetty | 4940718.92 | 425938.45 | ODEQ | LASAR | 08/08/2001 | 08/08/2001 |
| 25645 | Yaquina Bay mid-channel near US Coast Guard Res. | 4941797.92 | 424855.96 | ODEQ | LASAR | 08/09/2001 | 08/09/2001 |
| 29242 | South Beach 0.1 km West of Day Use Restroom | 4938970.88 | 415381.85 | ODHS | STORET | 10/02/2002 | 12/14/2004 |
| 29243 | Nye Beach @ Hallmark Resort 0.2 km South from bottom of stai | 4941992.52 | 415487.17 | ODHS | STORET | 10/02/2002 | 09/21/2004 |
| 29244 | Nye Beach 0.1 km West of stairs @ Hallmark Resort | 4942322.57 | 415566.05 | ODHS | STORET | 10/02/2002 | 09/21/2004 |
| 29245 | Nye Beach 0.3 km North of stairs @ Hallmark Resort | 4942718.53 | 415617.22 | ODHS | STORET | 10/02/2002 | 09/21/2004 |
| 29333 | South Beach Campground Trail between A & B loop | 4939483.01 | 415384.57 | ODHS | STORET | 10/02/2002 | 12/14/2004 |
| 29334 | South Beach Campground Trail C Loop | 4939822.03 | 415202.49 | ODHS | STORET | 10/02/2002 | 12/14/2004 |
| 29335 | Yaquina Bay State Park Beach-South | 4941249.44 | 415388.60 | ODHS | STORET | 10/02/2002 | 05/11/2004 |
| 29336 | Yaquina Bay State Park Beach-Middle | 4941433.93 | 415383.87 | ODHS | STORET | 10/02/2002 | 05/11/2004 |
| 29337 | Yaquina Bay State Park Beach-North | 4941659.16 | 415406.65 | ODHS | STORET | 10/02/2002 | 05/11/2004 |
| OR99-0024 | Yaquina Bay | 4941244.00 | 418052.15 | ODEQ | STORET | 08/17/1999 | 08/17/1999 |
| OR99-0025 | Yaquina River | 4938663.93 | 419496.70 | ODEQ | STORET | 08/18/1999 | 08/18/1999 |
| OR99-0026 | Yaquina River | 4935867.54 | 423630.97 | ODEQ | STORET | 08/18/1999 | 08/18/1999 |

ODEQ: Oregon Department of Environmental Quality; ODA: Oregon Department of Agriculture - Salem Lab; ODHS: Oregon Department of Human Services

## Permitted Discharges

The National Pollutant Discharge Elimination System (NPDES) Permitting Program sites in Yaquina Bay were obtained from ODEQ and mapped in Figure 38 for the bay and Figure 39 for the area near the City of Toledo outfall. Table 9 lists the NPDES sites closest to the discharge outfall and includes the City of Toledo's discharge.

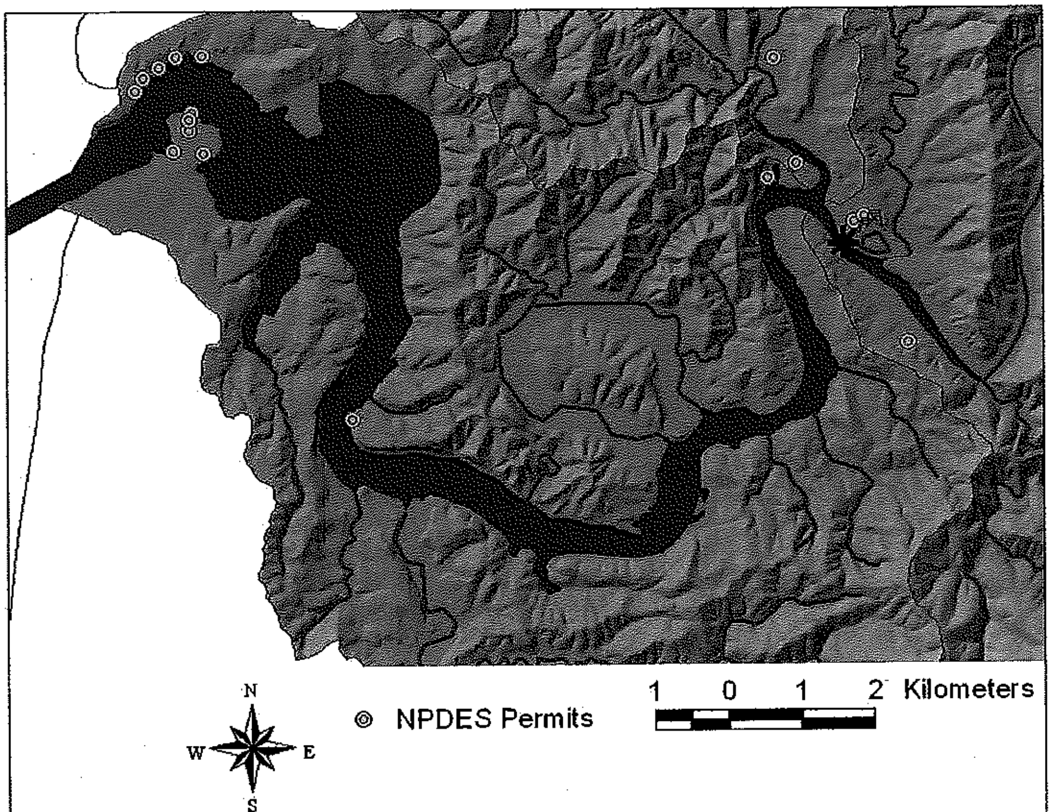

Figure 38: National Pollution Discharge Elimination System Permits in Yaquina Bay and River



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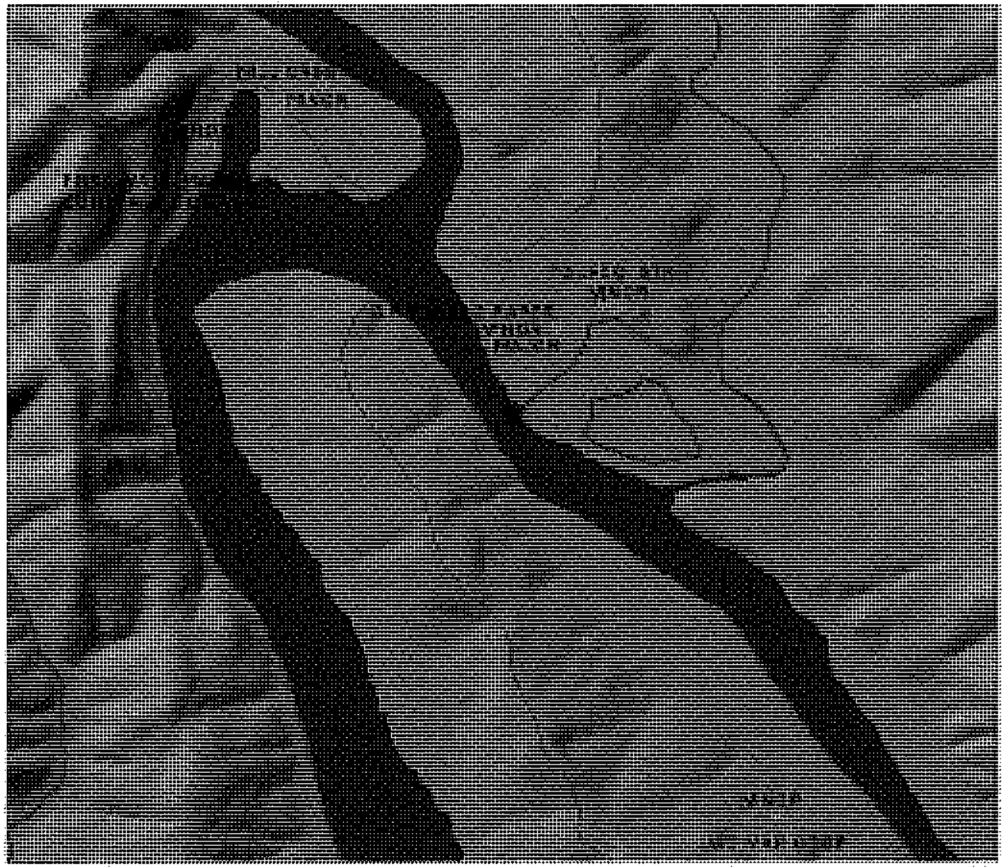

Figure 39: National Pollution Discharge Elimination System Permits near the City of Toledo WWTF discharge point



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**Table 9: NPDES Permit sites near the City of Toledo outfall.**

| File Number | Legal Name | Address | City | Category | Class | Latitude | Longitude | Type | LLID | River Mile | SubBasin Code |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 109703 | Fred Wahl Marine Construction, Inc. | 1000 Altree Lane | Toledo | STM | Minor | 44.6175 | 123.9479 | GEN12Z | 1240830 446097 | 12.50 | 17100204 |
| 32947 | Georgia-Pacific West, Inc., Toledo Paper | 1 Butler Bridge Rd | Toledo | STM | Minor | 44.6122 | 123.9330 | GEN12Z | 1239404 446146 | 0.20 | 17100204 |
| 32947 | Georgia-Pacific West, Inc., Toledo Paper | 1 Butler Bridge Rd | Toledo | IND | Major | 44.6122 | 123.9330 | NPDES-IW-A | 1240682 445993 | 99.00 | 17100204 |
| 110406 | Jac Mar Corporation | 1877 Elk City Rd | Toledo | STM | Minor | 44.5975 | 123.9235 | GEN12Z | 1240830 446097 | 15.00 | 17100204 |
| 111693 | Plum Creek Timberlands, L.P., Mill Creek Pit | 380 NW 1st St. | Toledo | STM | Minor | 44.6194 | 123.9430 | GEN12A | 1239404 446146 | 0.52 | 17100204 |
| 89103 | City of Toledo, STP | 1105 SE Fir St. | Toledo | DOM | Minor | 44.6129 | 123.9311 | NPDES-DOM-Da | 1240830 446097 | 10.20 | 17100204 |

## Shellfish Areas

The shellfish areas in Yaquina Bay were identified by updating a map from Furfari, S. A. (1985) based on conversation with the Oregon Department of Agriculture's Shellfish Program (Jim Johnson, Land Use and Water Planning Coordinator) and the Oregon Department of Fish and Wildlife (ODFW, Mitch Vance). The shellfish areas in Yaquina Bay are divided into two groups: the commercial oyster harvesting, which is overseen by the ODA Shellfish program and Recreational clamming, which is overseen by ODFW. Figure 40 shows a map of Yaquina Bay and the shellfish areas. The nearest recreational clam digging area is 350 m downstream of the of the City of Toledo outfall.

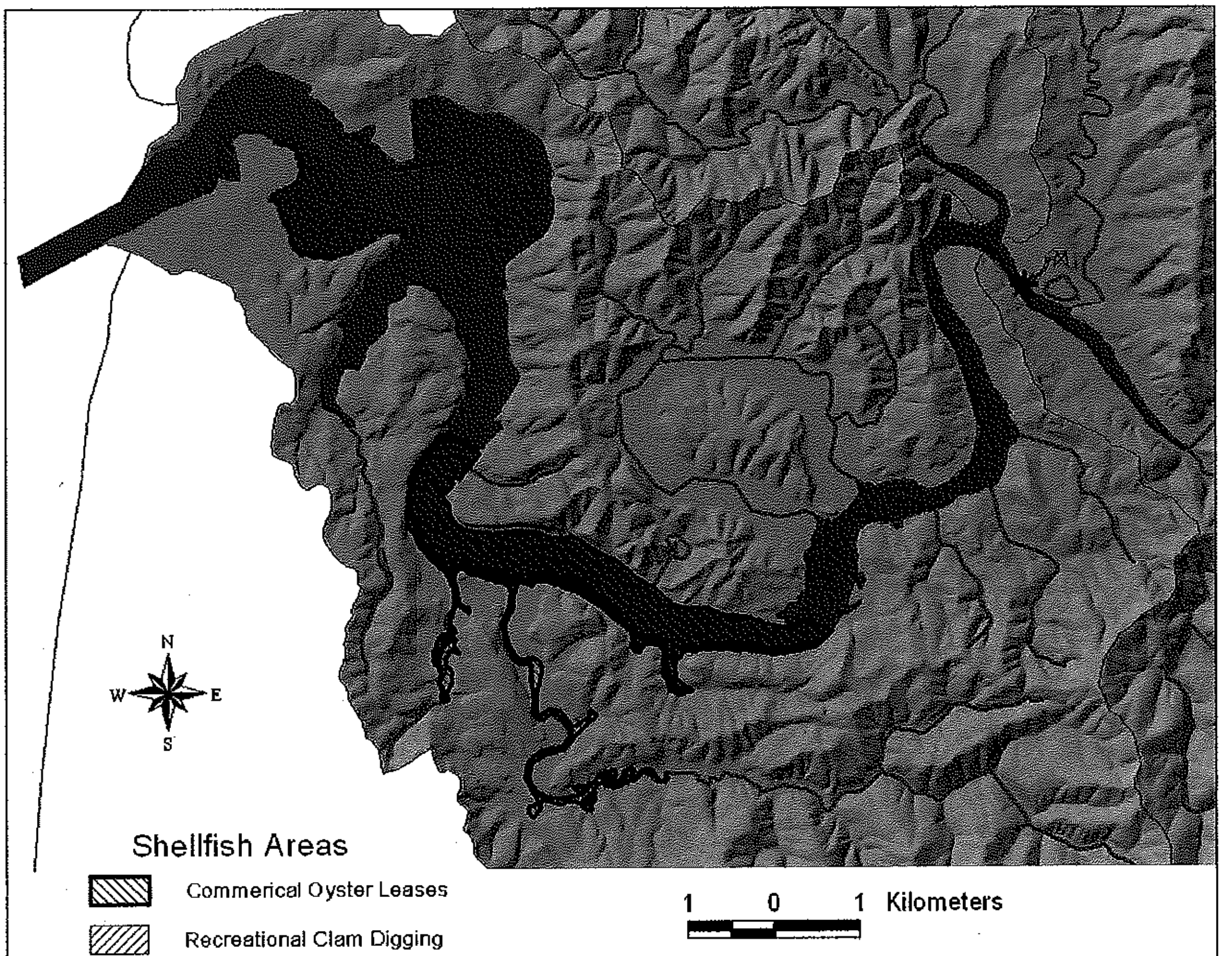


**Figure 40: Commercial Oyster and Recreational Clam digging areas in Yaquina Bay and River**

## Beach and Water Access

In addition to the shellfish areas public access and recreation sites were identified and mapped in Yaquina Bay based on data from the Oregon Department of Parks and Recreation. Figure 41 shows a map of the bay indicating sites for boat, pedestrian, vehicle and just visual access to the bay. The figure also includes shellfish areas. Although the public can access the Bay at many locations and travel upstream, there are no public access points near the City of Toledo outfall.

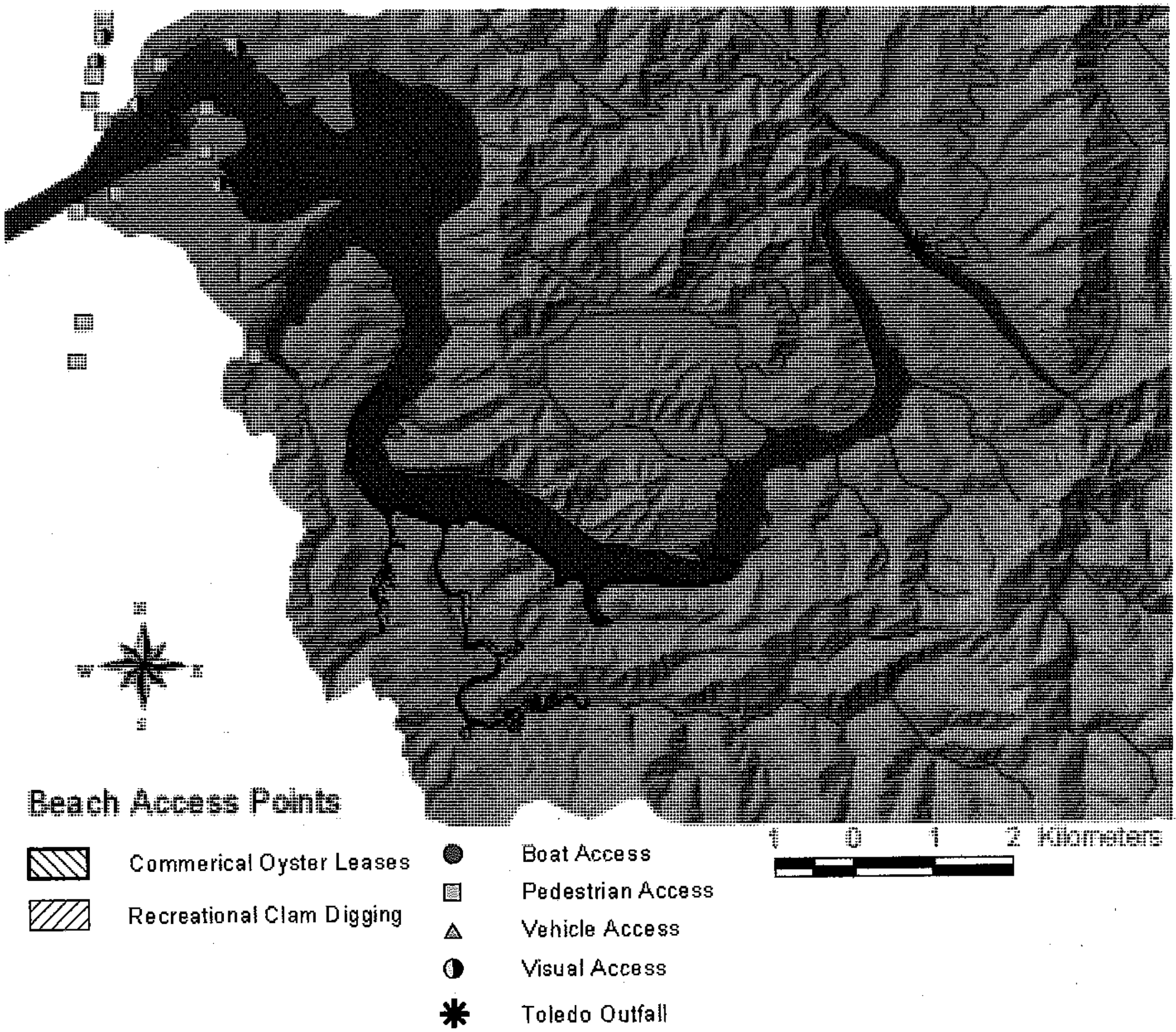


**Figure 41: Beach Access in the Yaquina Bay and River**

## Model Analyses

The modeling of dilution in the vicinity of the outfall was based on both an analytical model of the far field mixing assuming the plume was well-mixed vertically and had little momentum in its discharge as well as a CORMIX model of this discharge. The results from both were compared.

The objective of the modeling study is to assess compliance with the mixing zone established by the Oregon DEQ. The current mixing zone regulation is shown below:

Definition of dilution:

$$S\text{: dilution} = \frac{Q+Q_e}{Q}$$

Q: wastewater flow rate

$Q_e$: entrained flow rate

## *CE-QUAL-W2 Model of Yaquina River*

### CE-QUAL-W2 Model Set-up

The purpose of having a hydrodynamic model of the Yaquina River at the location of the outfall was to predict the depths, velocities, and salinities during critical periods of river flow (7Q10) and tidal height (low-low water). A model, CE-QUAL-W2 (Cole and Wells, 2004), was developed by EPA (Brown, 2005) of the Yaquina River from Elk City to the mouth of Yaquina Bay at South Beach as shown in Figure 42. The model was set-up for the 2002 calendar year and was developed in CE-QUAL-W2 Version 3.0.

The model would then be run for critical conditions:

- 7Q10 flow in the Yaquina River
- Month of September tidal conditions since the 7Q10 occurs in that month

The model would then be examined for extreme low-water conditions in the month and then parameters from this would be used to drive a near-field model of the Yaquina River.

The following is a list of model file changes for the CE-QUAL-W2 model received from the EPA on August 10, 2005:

- Converted the V3.0 model to V3.2. This involved redoing the control file, w2_con.npt, and bathymetry file (in V3.2, the shade information is in a separate input file) and generating new input files: graph.npt, wsc.npt, and shade.npt
- Many of the files were renamed to a simpler "*.npt" convention.
- Many input files had Julian day corrections made as a result of running the model preprocessor. These errors were a result of Julian days that were out of order in the input files.
- The concentration input files for tributaries and the branch inflow had their columns rearranged (for V3.2, TDS is always the 1st column)
- The downstream boundary condition concentration file had its number of active constituents reduced from 7 to 3 (TDS, water age, and tracer).
- All time series input files had their Julian days reordered. Julian day 1.5 corresponds to January 1 at 12:00 noon. Under the prior scheme that V3.0 was using, this would have been day 0.5.

In addition the bathymetry was also changed. Since the tidal prism above the Butler Bridge at Toledo is an important aspect of the modeling of this part of the river, the bathymetric segments of the CE-QUAL-W2 model were re-examined in more detail. According to EPA, the model grid at the Butler Bridge was continued all the way to Elk City (Figure 42). Tidal dynamics affecting water levels at Elk City. According to Goodwin et al. (1970) the head of tide is 137500 ft from the estuary mouth, whereas Elk City is 118,500 ft from the mouth. So there is some part of the Yaquina River that could be modeled further above Elk City to account for the full tidal prism impacts. (Note that Furfari (1984) claimed that the head of tide is approximately 5,000 ft above the Butler Bridge. Furfari (1984) performed a preliminary computation of the tidal prism starting 5,000 ft above this bridge and moving downstream. His estimates must be used with caution since his approach was not based on data.)

In order to improve on bathymetric representation of the Yaquina River above the Butler Bridge, a GIS map of the Yaquina River centerline was digitized as shown in Figure 43. The resulting profile or slope of the water surface is shown in Figure 44. This indicates although also very approximate, that the water slope is approximately 0.000279. We decided to apply that slope to the Yaquina River from the bridge to Elk City using the same channel shape as at Butler Bridge. This effectively lowers the tidal volume moving upstream from what the EPA model had assumed. It was this model that was used in the CE-QUAL-W2 model of the Yaquina River.

Also, according to Figure 45, the EPA model bottom thalweg elevations, relative to MLLW, were compared to recent channel bathymetric data relative to the datum NGVD29. According to this figure, it appears that the W2 model grid is actually in NGVD29 rather than MLLW. The conversion between the 2 elevations is:

$$\text{NGVD29, m} = \text{MLLW, m} - 1.2172\ \text{m}$$

Hence, the value of EBOT in the CE-QUAL-W2 control file, w2_con.npt was adjusted from -13.9 to -12.283. This also made the model bathymetry consistent with the downstream model forcing from the ocean, which was based on tidal data relative to MLLW. This also means that all outputs from the W2 model would be in MLLW and would need to be converted to NGVD29 outside the model.

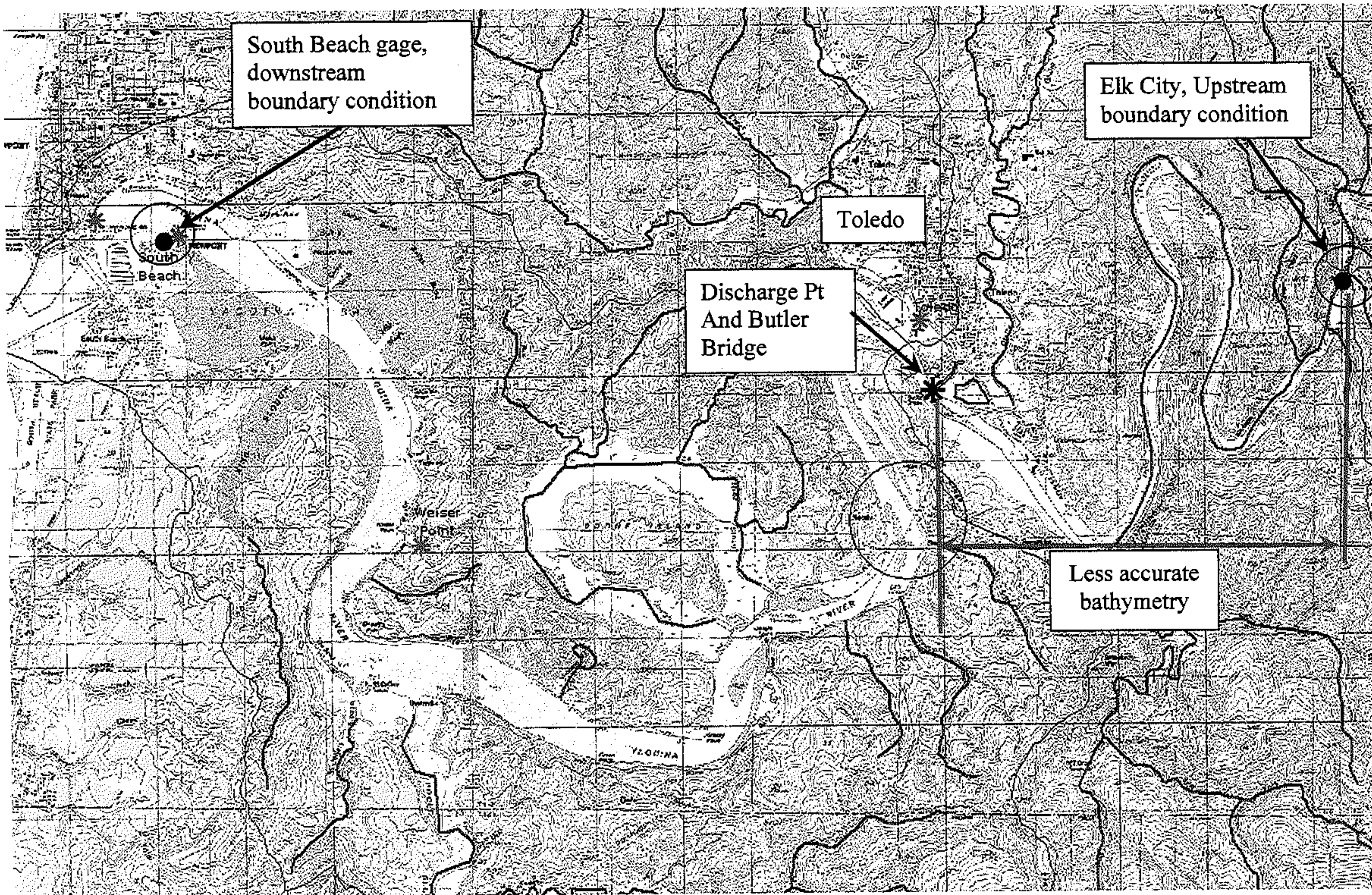


Figure 42: Geographical extent of the CE-QUAL-W2 model of Yaquina River from EPA (Brown, 2005).

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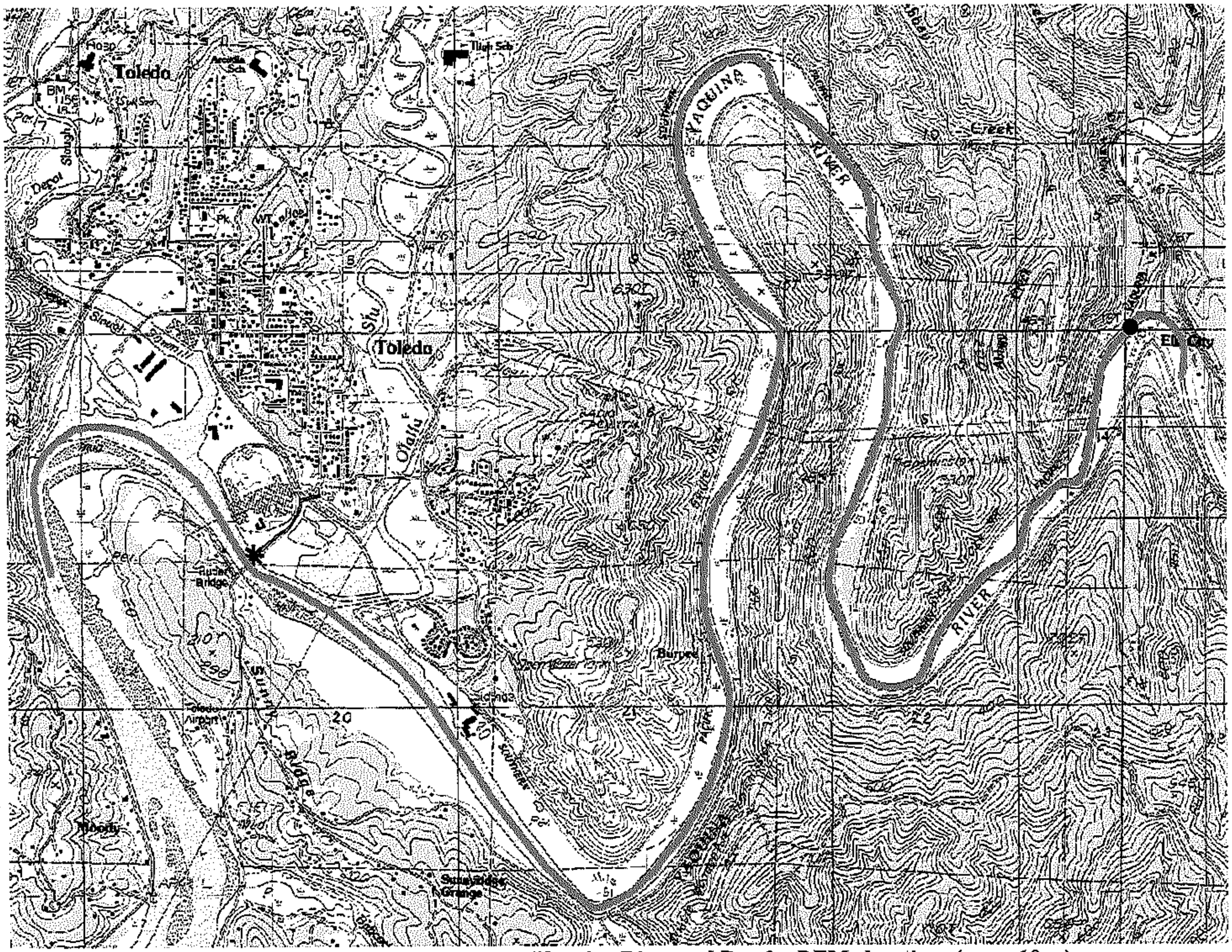


Figure 43: Centerline points along the Yaquina River and Bay for DEM elevations (every 10 m)

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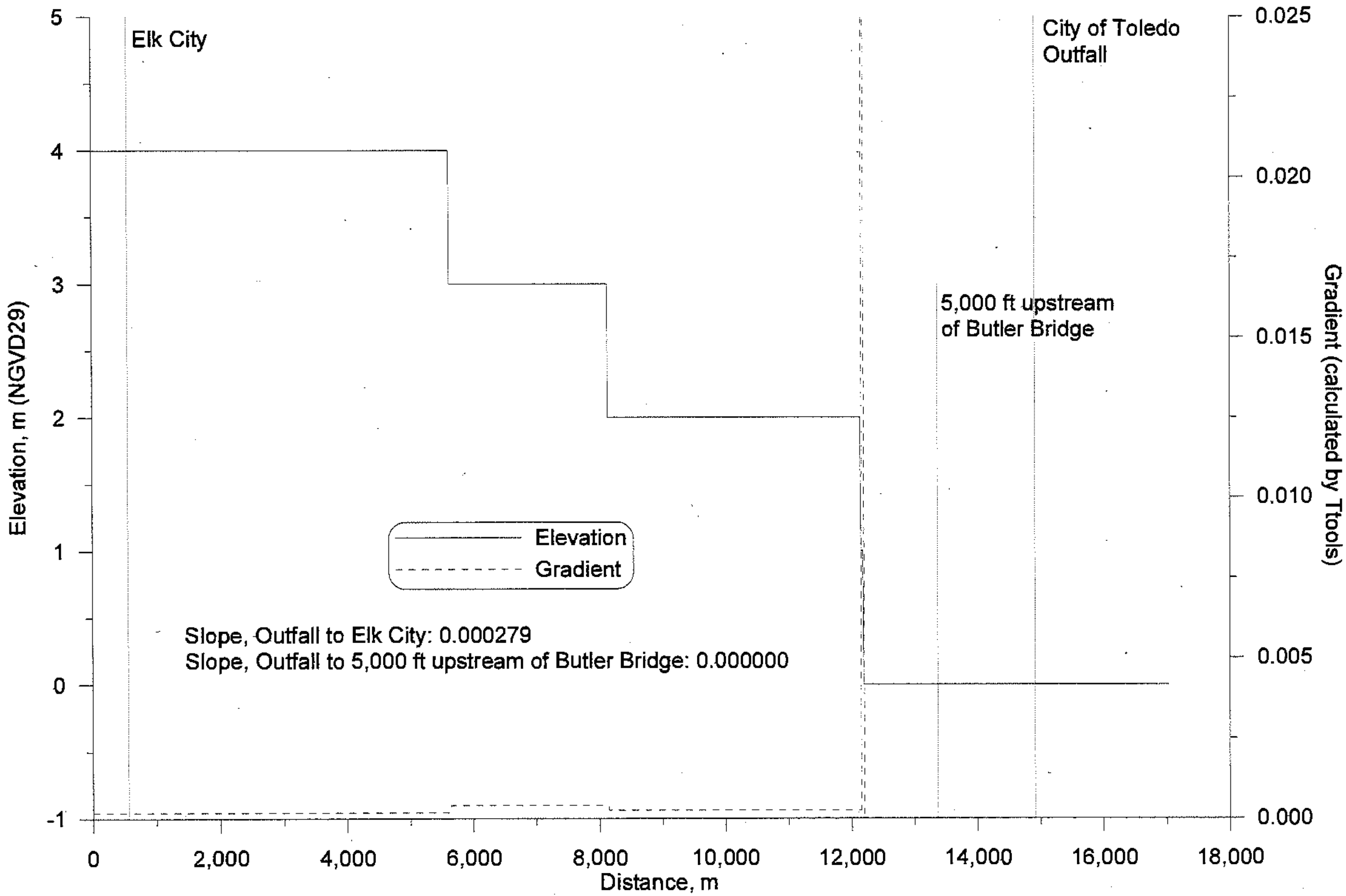


**Figure 44: Longitudinal Elevation Profile along Yaquina River and Bay**

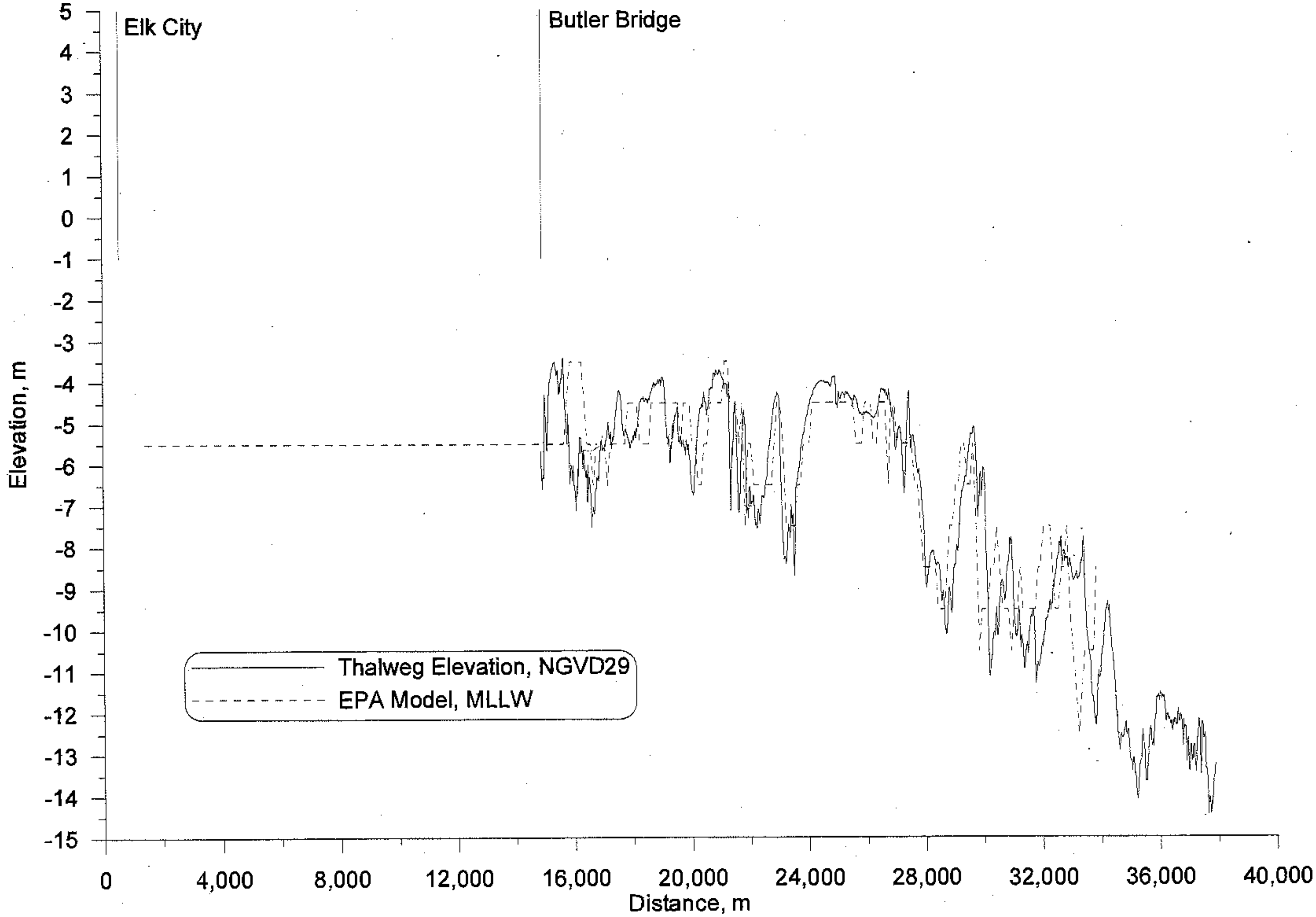


**Figure 45: Elevations along thalweg from CE-QUAL-W2 model received from EPA and from recent soundings data.**



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### CE-QUAL-W2 Model Results

The CE-QUAL-W2 model was run for the month of September with a constant 7Q10 flow of 0.24 $m^3/s$ with the existing tidal dynamics and City of Toledo inflows for 2002. Critical points in the tidal cycle were evaluated with the CE-QUAL-W2 model. Table 10 shows an evaluation of model predictions at LW (low water), HW (high water), HHW (high-high-water) and LLW (low-low-water) for critical low water conditions at the outfall (model segment 146). Figure 46 describes the definitions of depths listed in Table 10. These results for the entire month of September are shown in Figure 47 and the time period of interest is shown more clearly in Figure 48. A frequency curve of the water temperature predictions in the river at the outfall is shown in Figure 49. A frequency curve for depth-average velocities is shown in Figure 50. These results are the basis for the near field mixing modeling presented in the next section.

**Table 10: Typical CE-QUAL-W2 widths, velocities, densities, and depths at four points in a typical tidal cycle for lowest water and 7Q10 flow from the Yaquina River at the location of the City of Toledo outfall.**

| Julian day | Tidal period | Density, $g/m^3$ and TDS, ppt | Depth-averaged cross-sectional velocity, m/s | Cross-sectional area at outfall, $m^2$ | Top width at outfall, m | Maximum depth, m | Average depth, m (see Figure 46Figure 46) |
|---|---|---|---|---|---|---|---|
| 249.75 | LW | 1009.01/13.6 | 0.256 | 261 | 80 | 4.7 | 3.3 |
| 249.53 | HW | 1014.96/20.9 | 0.090 (essentially slack tide) | 286 | 82 | 6.9 | 3.5 |
| 249.99 | HHW | 1015.78/22 | -0.082 (essentially slack tide) | 600 | 82 | 7.3 | 7.3 |
| 250.30 | LLW | 1007.84/12.1 | 0.325 | 171 | 67 | 3.9 | 2.5 |

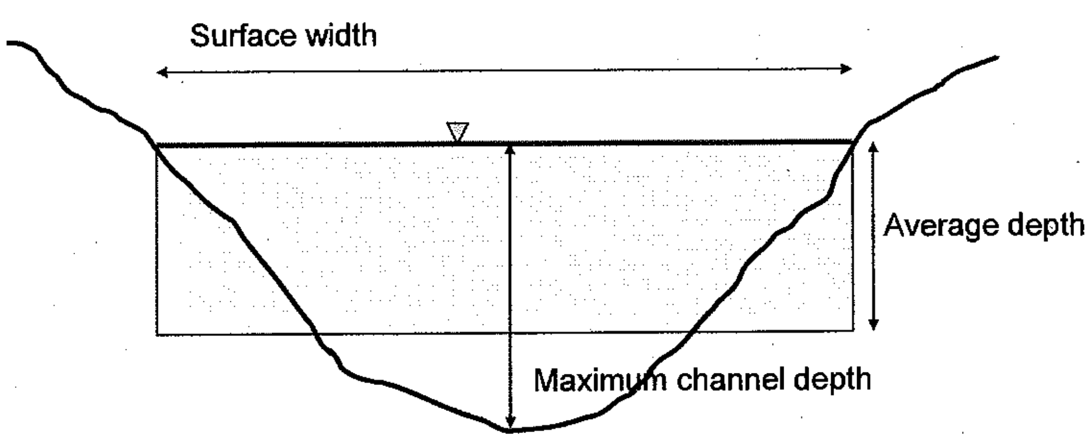


**Figure 46: Definition sketch for parameters in average depth.**

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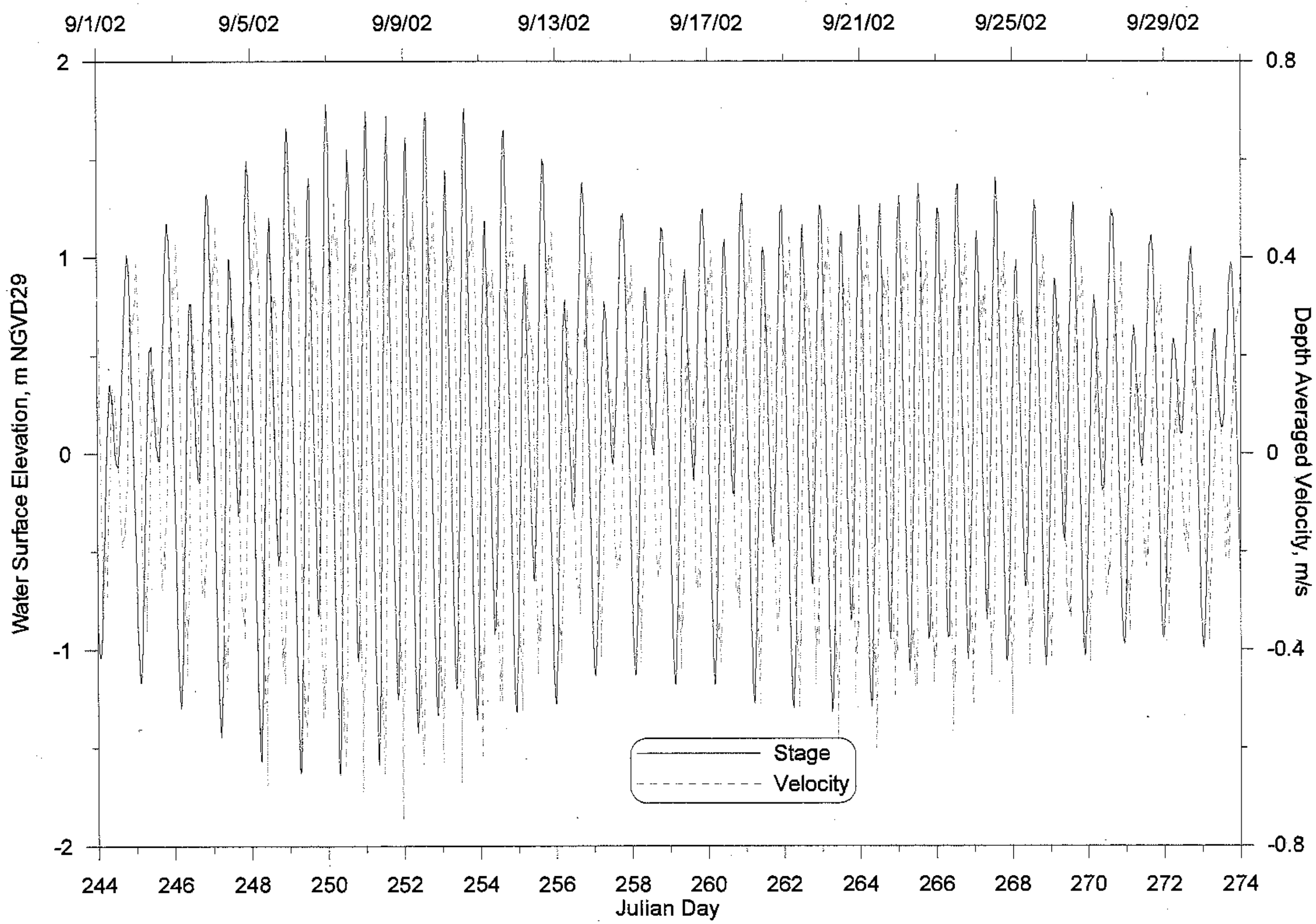


**Figure 47: Tidal stages and depth averaged velocity predictions at the City of Toledo outfall on the Yaquina River, September, 2002.**

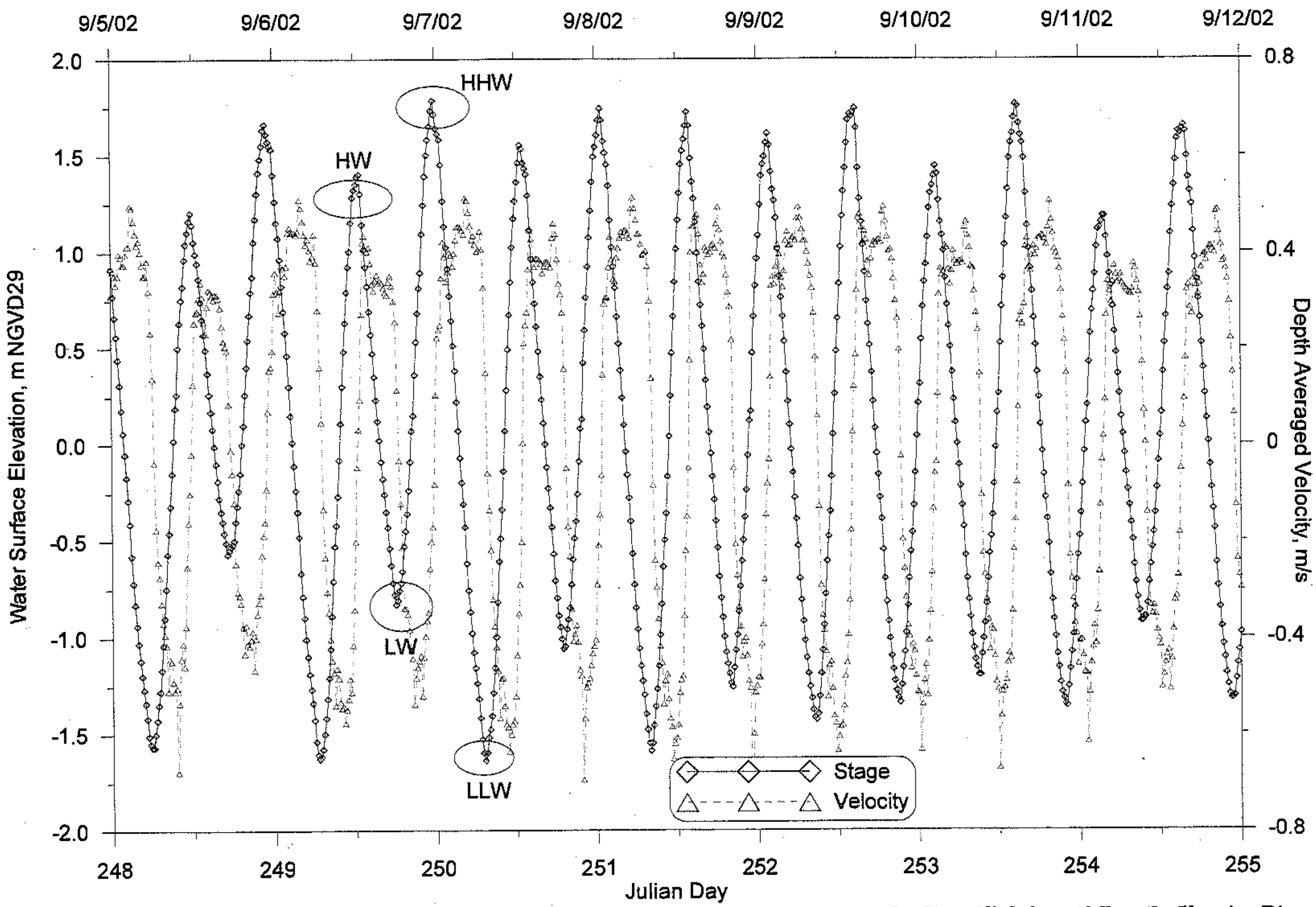


**Figure 48: Tidal stages and depth averaged velocity predictions during the largest tidal cycles at the City of Toledo outfall on the Yaquina River, September, 2002**

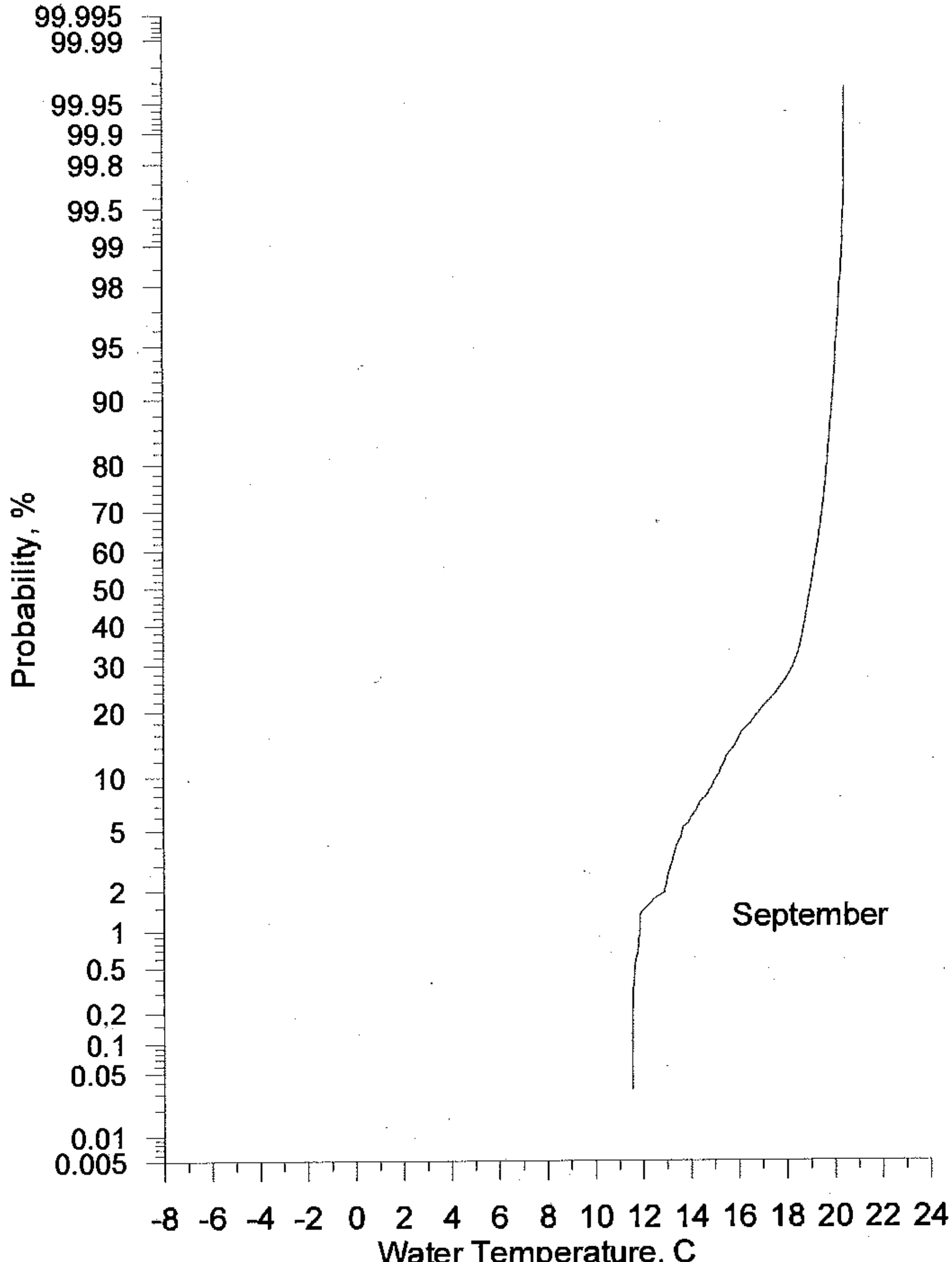


**Figure 49: Model predicted water temperature frequency in Yaquina River/Bay at City of Toledo outfall (Model segment 146)**

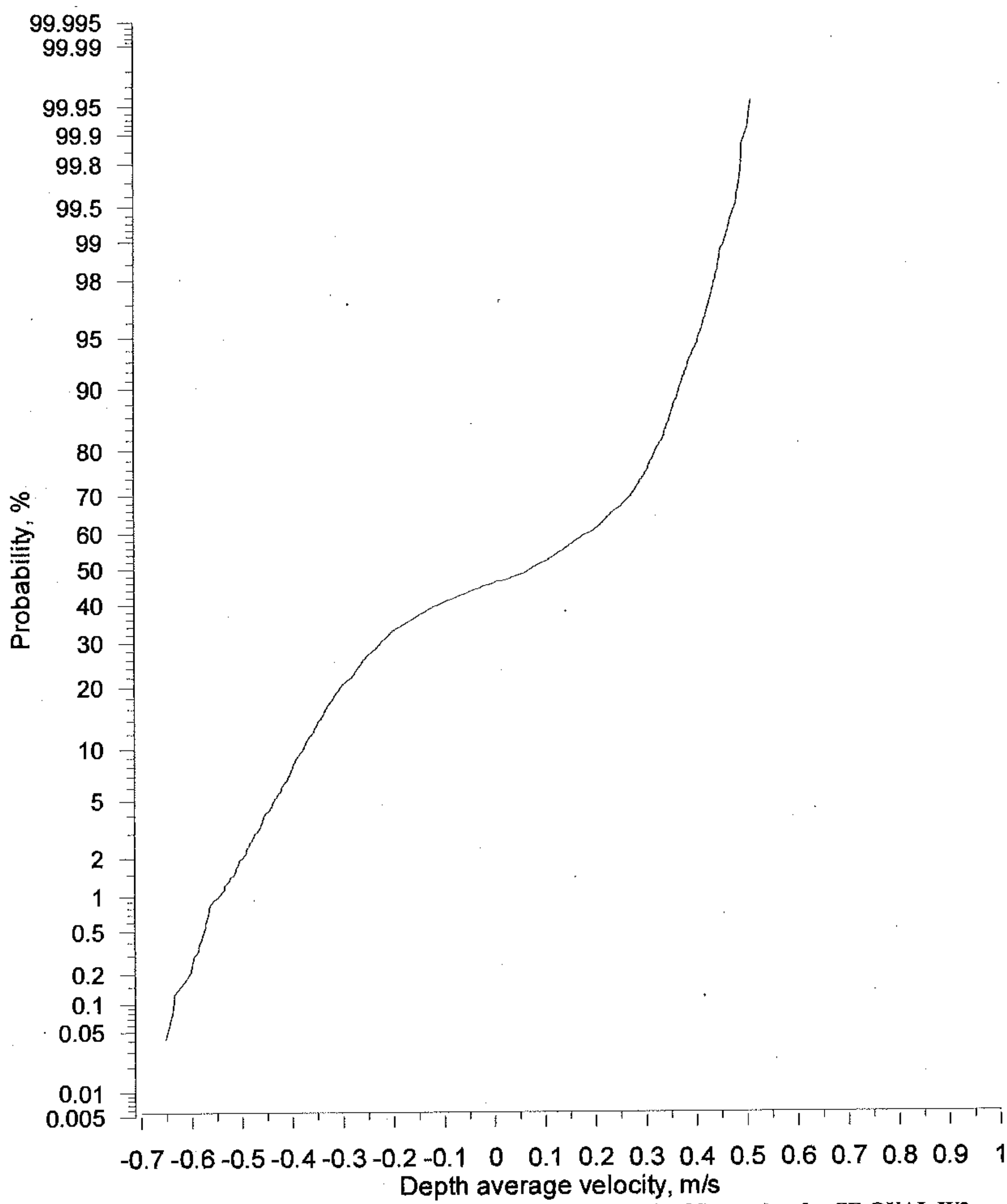


**Figure 50. Probability distribution of depth average velocity during month of September for CE-QUAL-W2 model segment 146.**

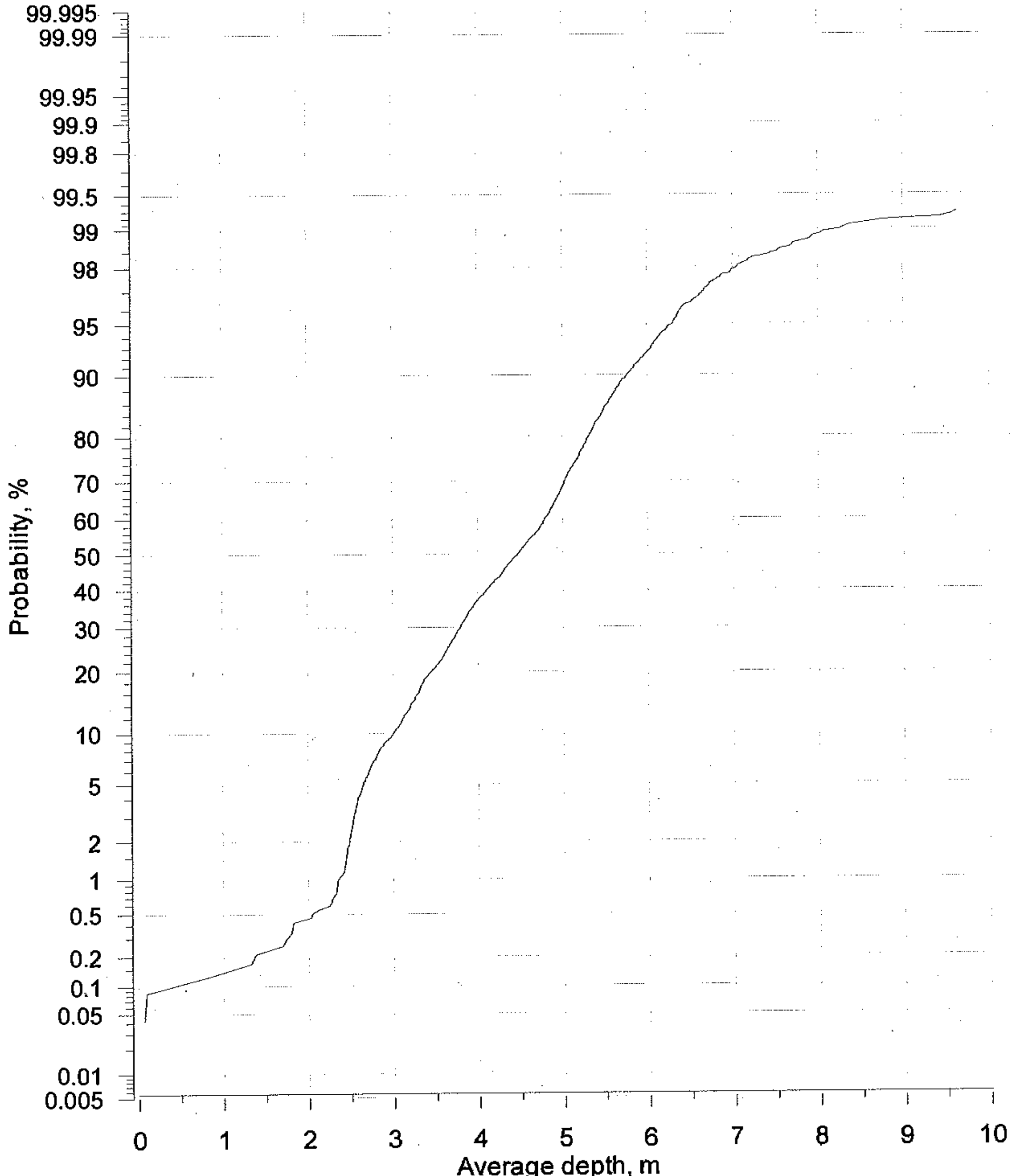


**Figure 51. Probability distribution of average depth during the month of September for CE-QUAL-W2 model segment 146.**

### *Analytical Model Scenarios and Results*

The governing equation for the far-field mixing plume is given by

$$\frac{\partial c}{\partial t}+u\frac{\partial c}{\partial x}+v\frac{\partial c}{\partial y}+w\frac{\partial c}{\partial z}=E_x\frac{\partial^2 c}{\partial x^2}+E_y\frac{\partial^2 c}{\partial y^2}+E_z\frac{\partial^2 c}{\partial z^2}-Kc$$

where c is the concentration; u, v, and w are the velocities in x, y, z; $E_x$, $E_y$, and $E_z$ are the turbulent diffusion coefficients in x, y, and z, respectively, and K is a first order decay coefficient.

Assuming that we have a steady-state discharge, neglect longitudinal diffusion, and assume the plume is well-mixed vertically and that v=w=0, the governing equation then becomes:

$$\bar{u}\frac{\partial \bar{c}}{\partial x} = E_y\frac{\partial^2 \bar{c}}{\partial y^2}-K\bar{c}$$

where the overbars imply that the state variable is vertically averaged. The solution to this approximate form is

$$\bar{c}\cong\frac{q'}{\sqrt{4\pi xUE_y}}\ \exp-\left[\frac{y^2U}{4E_yx}+\frac{xK}{U}\right]48$$

where $q'=\frac{QC_o}{h}$, Q is the flow rate, Co is the initial concentration, and h is the depth. The above solution assumes infinite boundaries laterally. If a channel is bounded by side walls with a width W and depth h, the solution using superposition is given by:

$$\bar{c}=\frac{q'}{UW}\frac{1}{\sqrt{4\pi x'}}\sum_{n=-\infty}^{\infty}\left(\exp\left[-\frac{(y'-2n-y_o')^2}{4x'}\right]+\exp\left[-\frac{(y'-2n+y_o')^2}{4x'}\right]\right)$$

where $x'=\frac{xE_y}{UW^2}$ and $y'=\frac{y}{W}$ and $y_o$ is the location of the source with y=0 defined as being at the bank. For the discharge on the side of the Yaquina River, the solution would be taking into account the reflective boundary condition of the channel:

$$\bar{c}\cong\frac{2q'}{\sqrt{4\pi xUE_y}}\ \exp-\left[\frac{y^2U}{4E_yx}+\frac{xK}{U}\right]$$

The lateral diffusion coefficient was estimated from turbulence theory as $E_y=0.6u_*h$ where $u_*$ is the shear velocity and h is the depth of the water. The shear velocity can be estimated as approximately 10% of the mean velocity, i.e., $u_*$=0.1u.

Assuming the most conservative conditions: the least dilution along the longitudinal axis of the flow (y=0) and no decay (K=0), the dilution as a function of distance, x, from the outfall (+ is downstream and – is upstream):

$$\frac{c}{C_o}=\frac{2Q}{h\sqrt{4\pi xUE_y}}.$$

Using the above approach, analytical model results for the expected concentration and dilution for a discharge of 1.0 MGD is shown in Figure 52 and Figure 53, respectively. Additionally, analytical model results for the expected concentration and dilution for a discharge of 0.5 MGD is shown in Figure 54 and Figure 55, respectively.

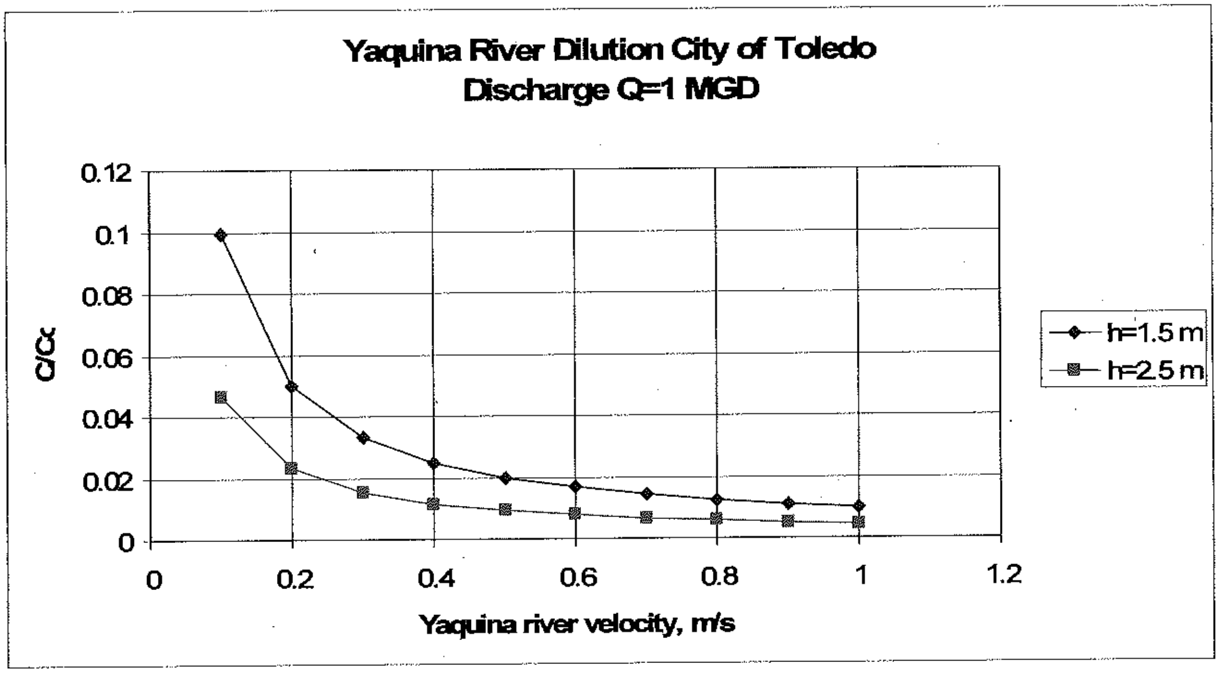


**Figure 52: Analytical model results of concentration at the edge of the 100 ft mixing zone as a function of river velocity at a discharge of 1 MGD.**

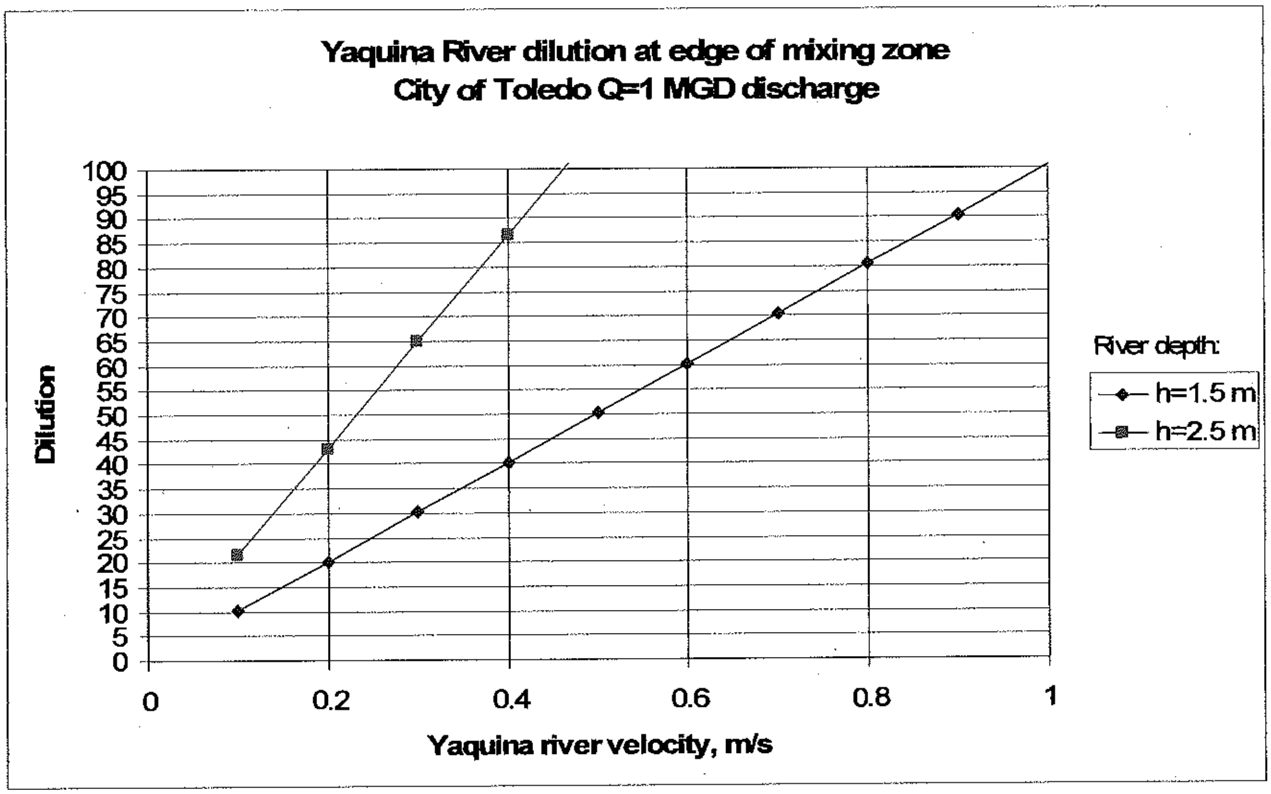


**Figure 53: Analytical model results of dilution at the edge of the 100 ft mixing zone as a function of river velocity at a discharge of 1 MGD.**

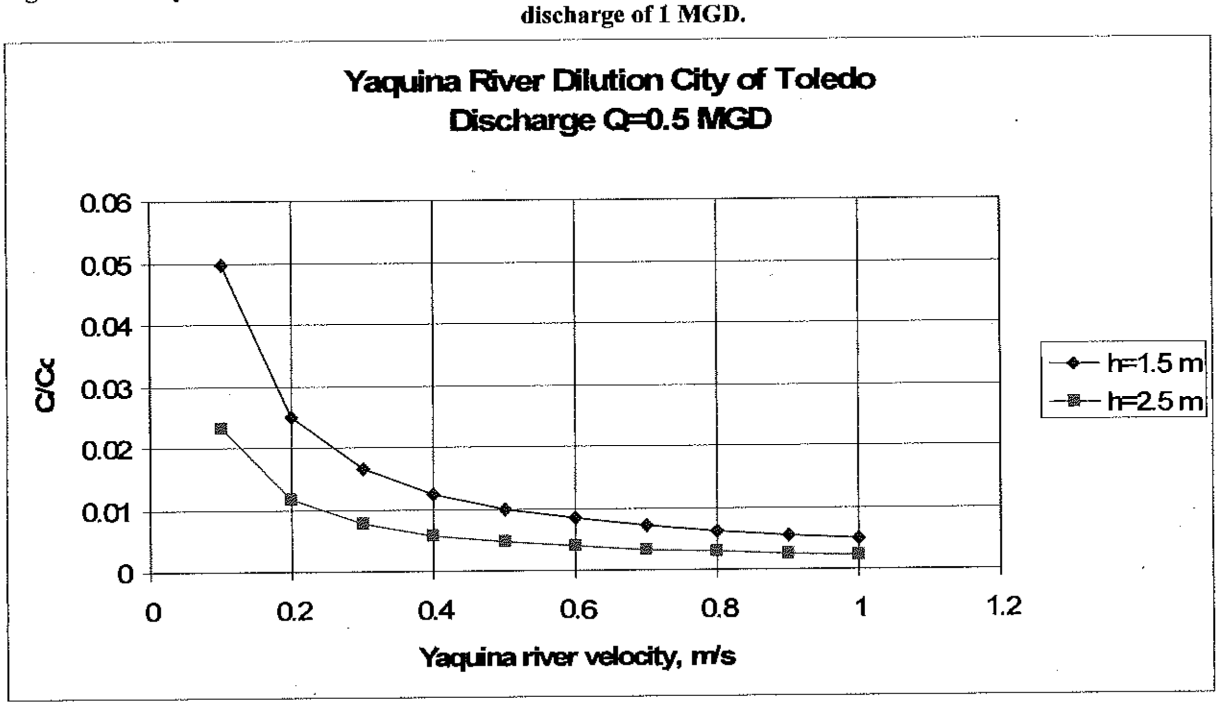


**Figure 54: Analytical model results of concentration at the edge of the 100 ft mixing zone as a function of river velocity at a discharge of 0.5 MGD.**

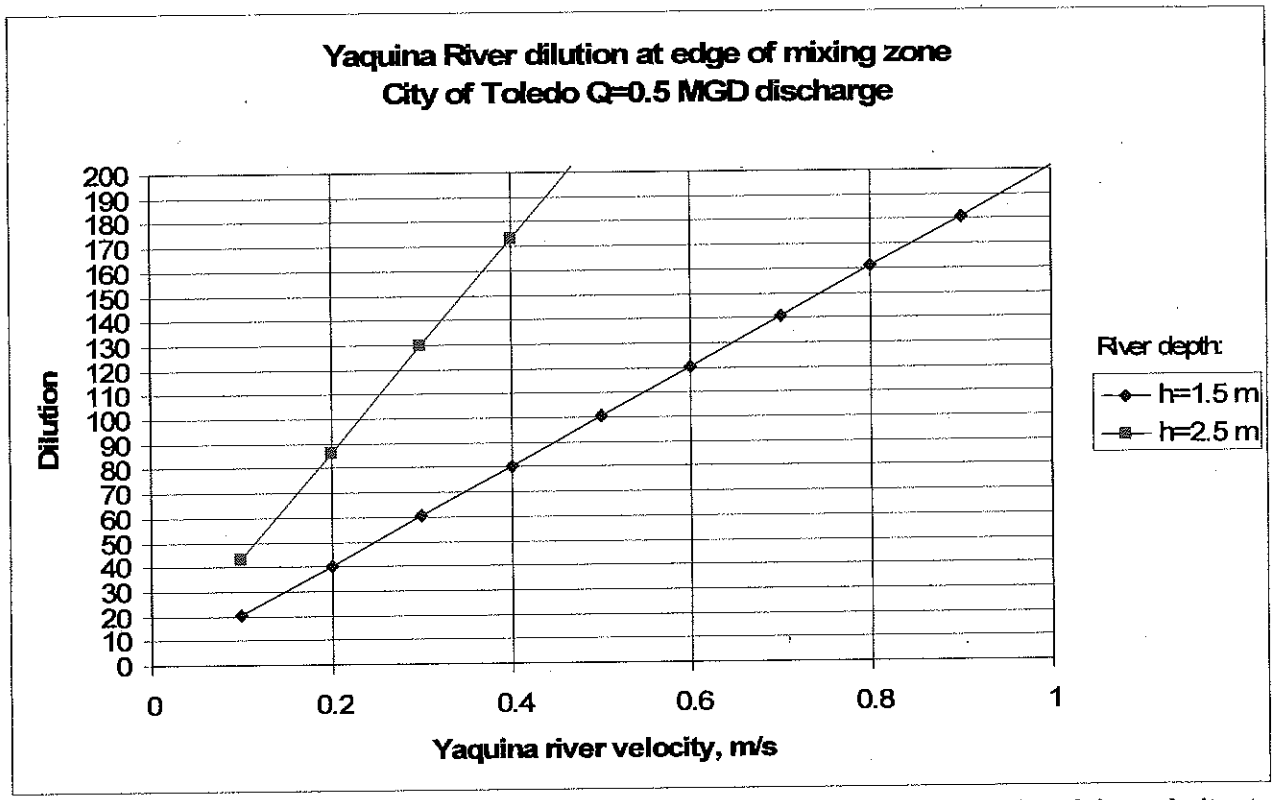


**Figure 55: Analytical model results of dilution at the edge of the 100 ft mixing zone as a function of river velocity at a discharge of 0.5 MGD.**

This particular model assumes a rectangular channel shape (see Figure 46) and a well-mixed vertical plume over that depth at the point of discharge for flow rates of 0.5 and 1.0 MGD from the City of Toledo. The mixing predicted with this model are expected then to be conservative because of this assumed initial vertical mixing and shape of the channel. For the most extreme case at LLW where the river velocity would be 0.325 m/s at a depth of about 2.5 m, the predicted dilution would be approximately 65 at 1.0 MGD and 130 at 0.5 MGD. It is expected that the actual dilution would be less than these values as shown in the CORMIX results.

## *CORMIX Model Scenarios and Results*

### CORMIX Model Set-up

The CORMIX model (EPA, 1996) was set-up to evaluate a surface discharge using CORMIX3 for the 4 points in the tidal cycle defined in Figure 48 during critical 7Q10 Yaquina River flows and low-water conditions. CE-QUAL-W2 model output was used to characterize the river for each of 4 points in the tidal cycle and the effluent discharge was set at 0.5 and 1.0 MGD, resulting in 8 model simulations.

Appendix 3 lists the CORMIX model input data and model results for the 4 tidal points with the effluent discharge at 1.0 MGD. Appendix 4 lists the CORMIX model input data and model results for the 4 tidal points with the effluent discharge at 0.5 MGD. All simulations used a release of 100 ppm of conservative dye at the point of discharge.

**CORMIX Model Results**

Figure 56 shows the dye concentration and dilution for an effluent discharge of 1.0 MGD at low low water in the tidal cycle. Figure 57 shows the dye concentration and dilution for an effluent discharge of 1.0 MGD at low water in the tidal cycle. Figure 58 shows the dye concentration and dilution for an effluent discharge of 1.0 MGD at high water. Figure 59 shows the dye concentration and dilution for an effluent discharge of 1.0 MGD at high high water. Figure 60 shows the dye concentration for the four points in the tidal cycle together for comparison and Figure 61 shows the dilution for the four points in the tidal cycle. Table 11 summarizes the dilution at the edge of mixing zone and at twice the distance of to the edge of the mixing zone. The results indicate the highest dilution occurs under low low water conditions but there is not much variability between the points in the tidal cycle.

**Table 11: Dilution ratios for City of Toledo effluent flow of 1.0 MGD for various stages of the tidal cycle**

| Downstream distance, ft | Low Low Water | Low Water | High Water | High High Water |
|---|---|---|---|---|
| 100 | 4.8 | 4.5 | 3.8 | 3.8* |
| 200 | 6.5 | 5.8 | 4.5 | 4.3* |
| *During high high water tidal flows were going upstream so distances are upstream for this scenario. | | | | |

Figure 62 shows the dye concentration and dilution for an effluent discharge of 0.5 MGD at low low water in the tidal cycle. Figure 63 shows the dye concentration and dilution for an effluent discharge of 0.5 MGD at low water in the tidal cycle. Figure 64 shows the dye concentration and dilution for an effluent discharge of 0.5 MGD at high water. Figure 65 shows the dye concentration and dilution for an effluent discharge of 0.5 MGD at high high water. Figure 66 shows the dye concentration for the four points in the tidal cycle together for comparison and Figure 67 shows the dilution for the four points in the tidal cycle. Table 12 summarizes the dilution at the edge of mixing zone and at twice the distance of to the edge of the mixing zone. These results indicate the highest dilution occurs under low low water conditions and there is a little more variability between the points in the tidal cycle than with a 1.0 MGD discharge.

**Table 12: Dilution ratios for City of Toledo effluent flow of 0.5 MGD for various stages of the tidal cycle**

| Downstream distance, ft | Low Low Water | Low Water | High Water | High High Water |
|---|---|---|---|---|
| 100 | 7.7 | 5.8 | 4.1 | 4.3* |
| 200 | 11.5 | 8.3 | 5.2 | 5.0* |
| *During high high water tidal flows were going upstream so distances are upstream for this scenario. | | | | |

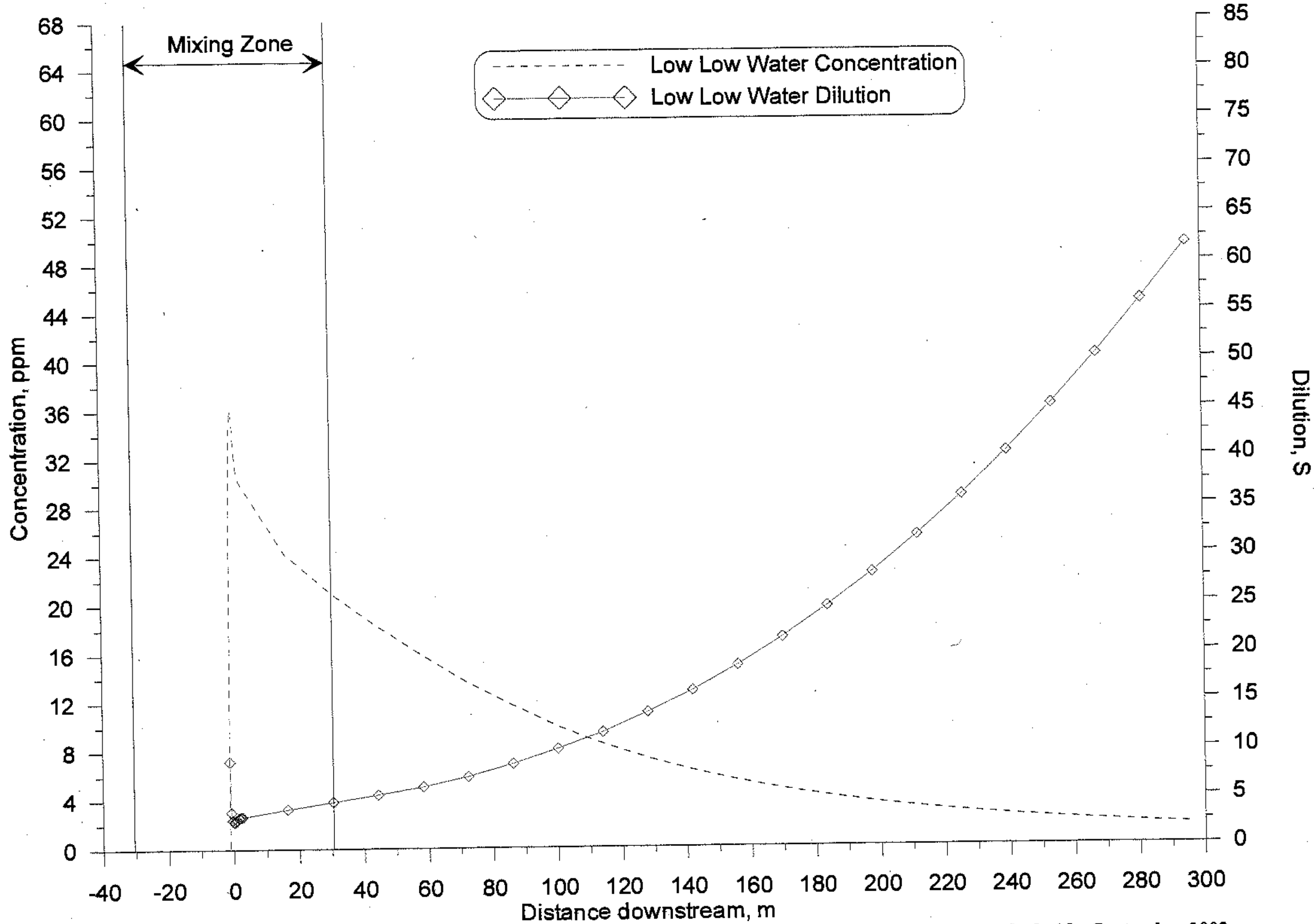


**Figure 56: Dye concentration and dilution for City of Toledo discharge of 1.0 MGD, during low low water slack tide, September 2002.**

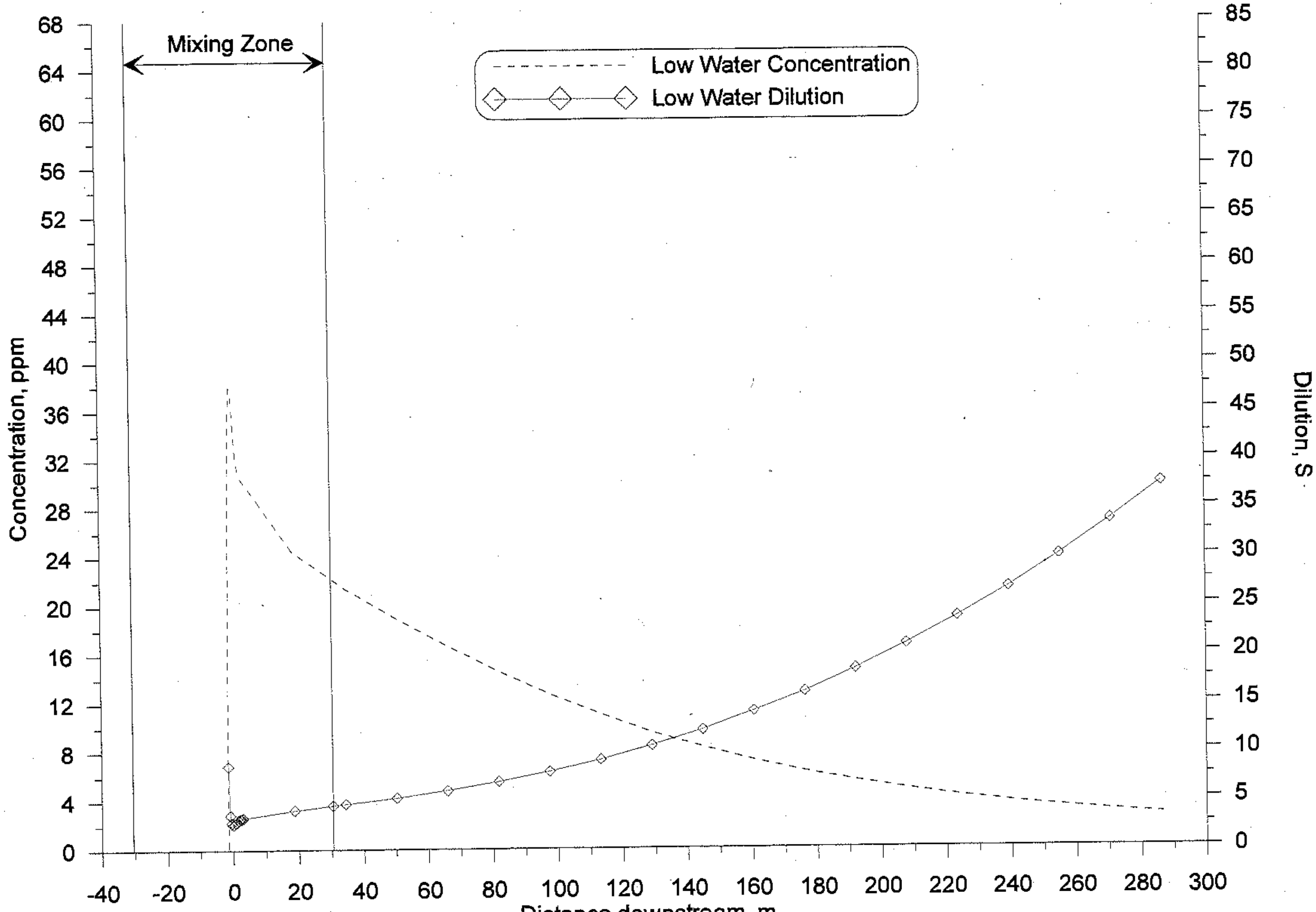


**Figure 57: Dye concentration and dilution for City of Toledo discharge of 1.0 MGD, during low low water slack tide, September 2002.**

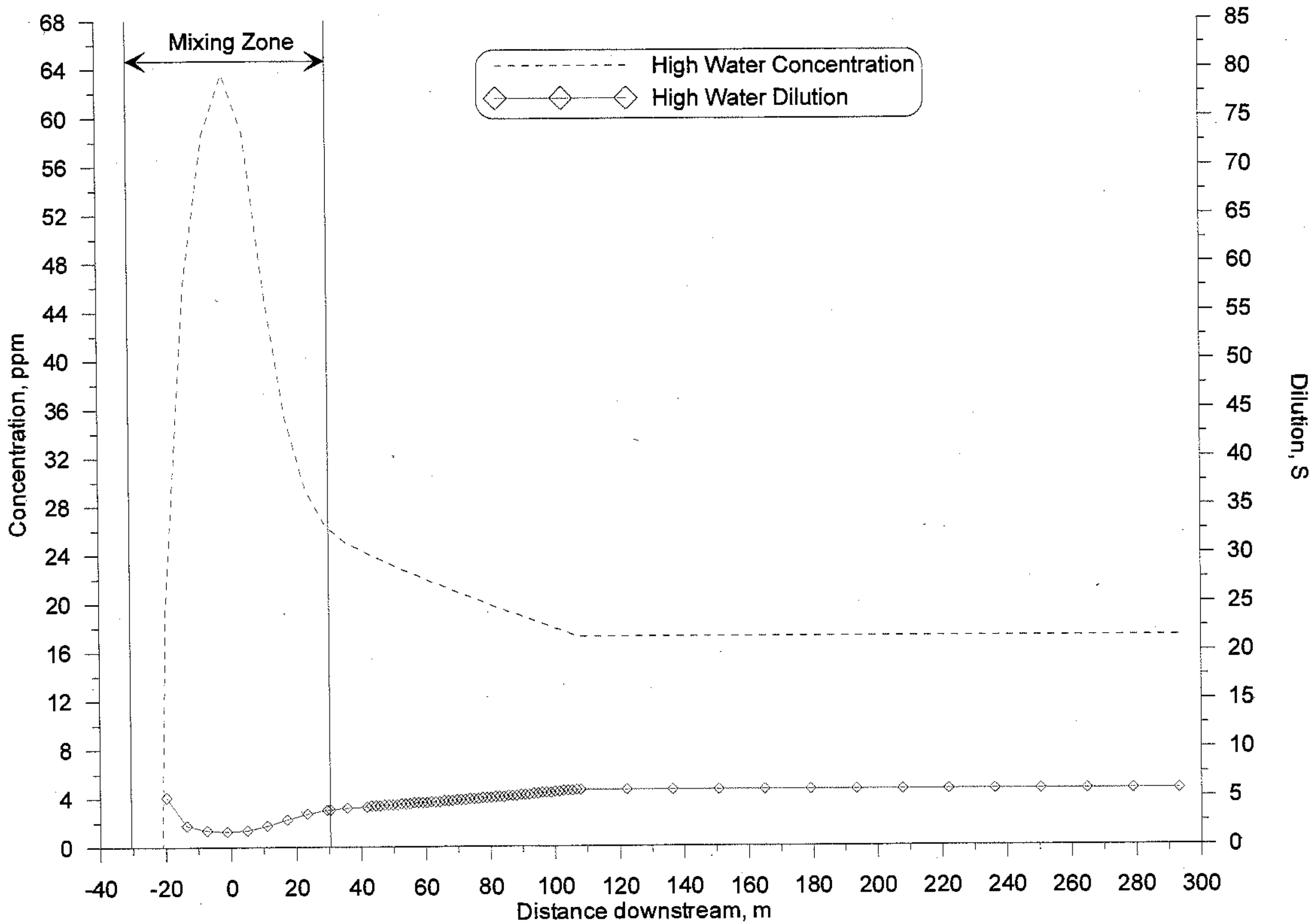


**Figure 58: Dye concentration and dilution for City of Toledo discharge of 1.0 MGD, during high water slack tide, September 2002.**

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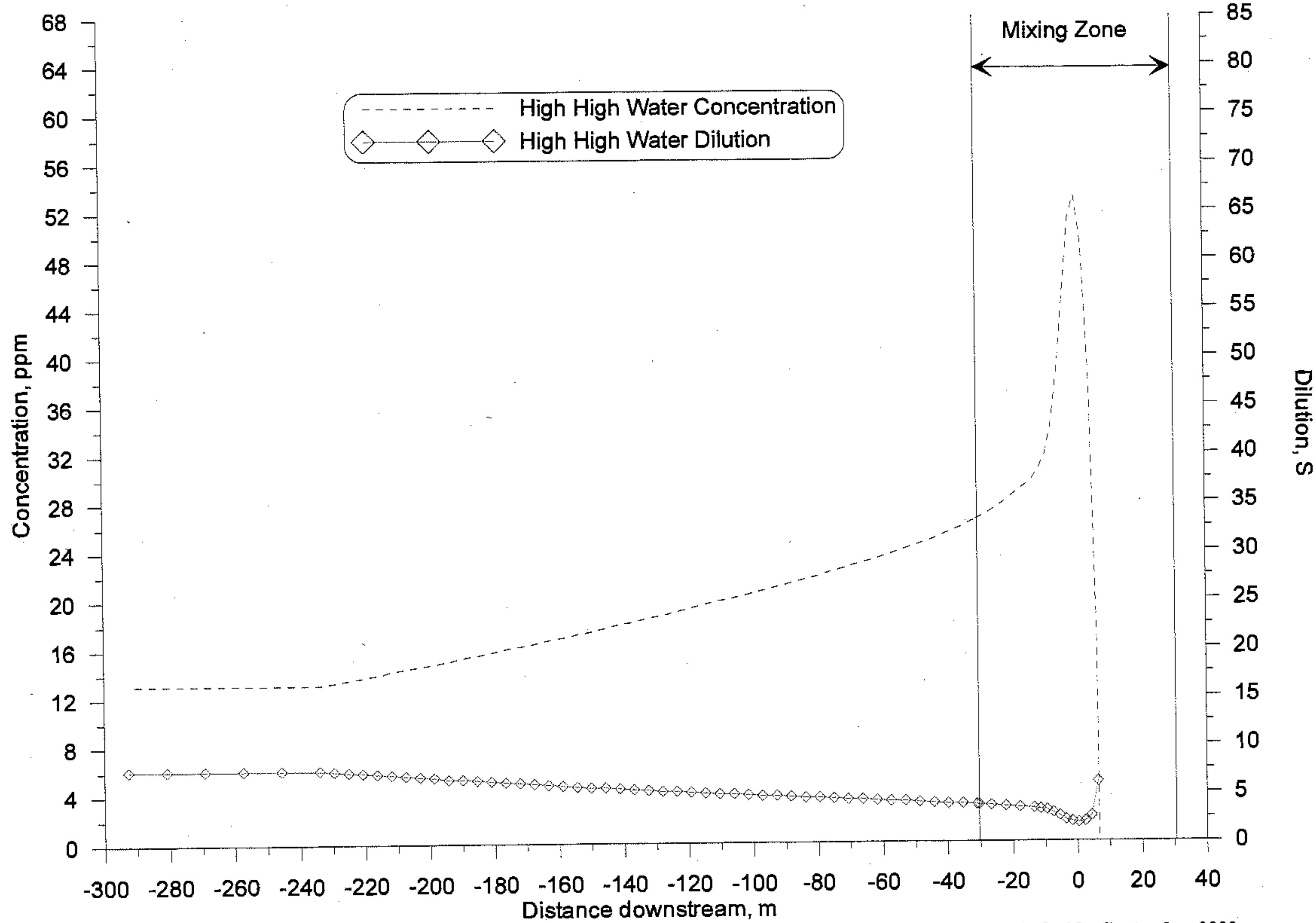


**Figure 59: Dye concentration and dilution for City of Toledo discharge of 1.0 MGD, during high high water slack tide, September 2002.**

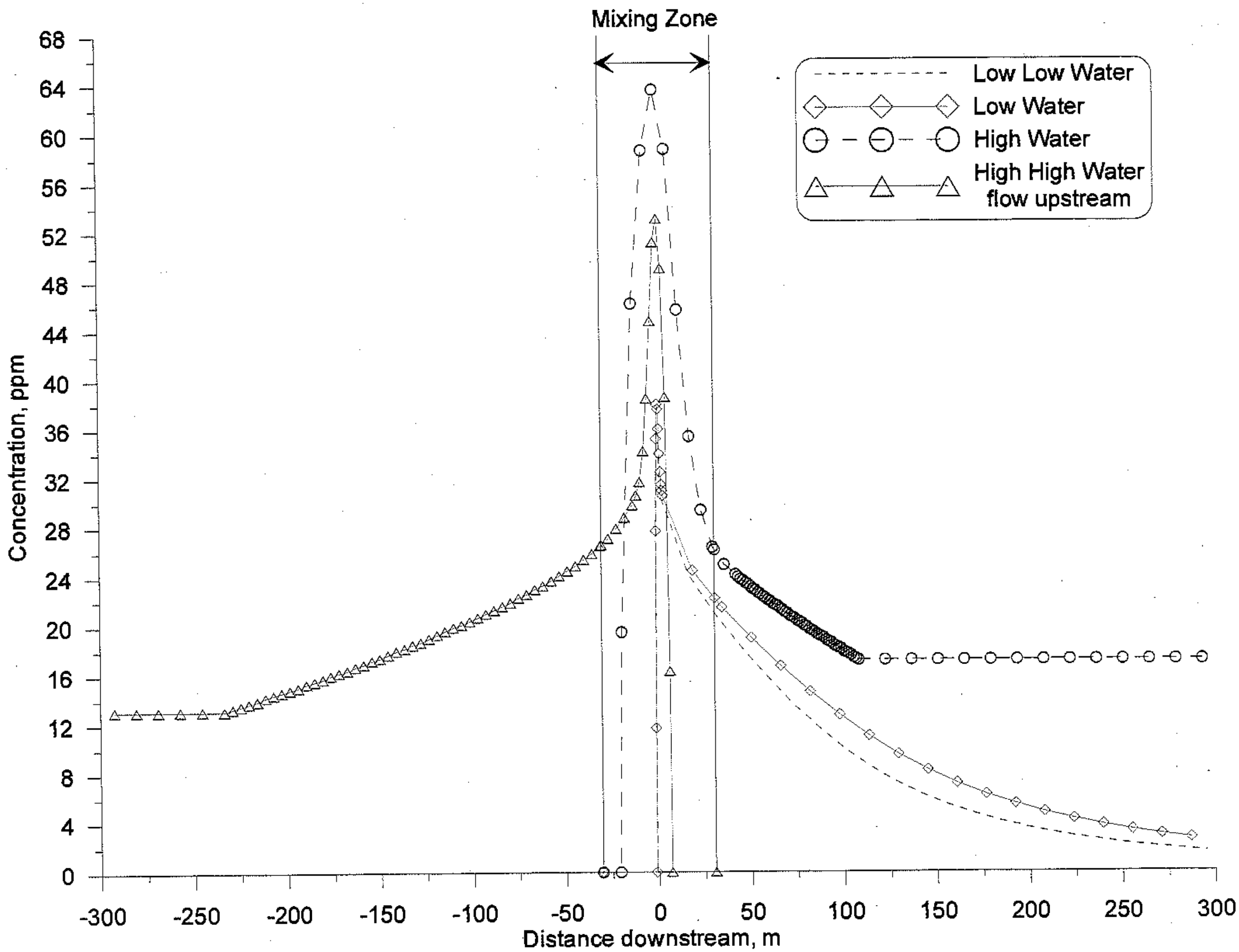


**Figure 60: Dye concentrations for City of Toledo discharge of 1.0 MGD, during various slack tides, September 2002.**



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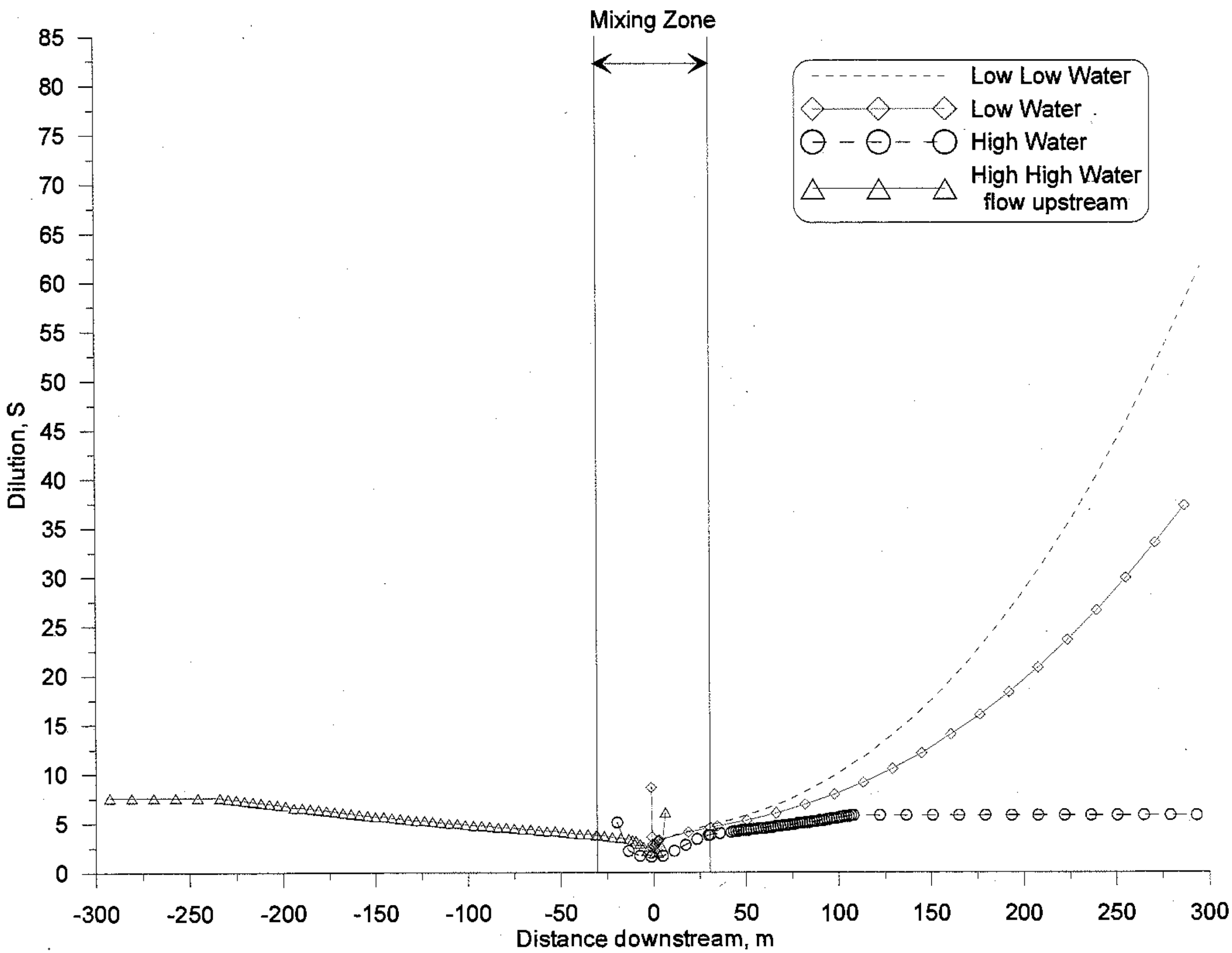


**Figure 61: Dye dilutions for City of Toledo discharge of 1.0 MGD, during various slack tides, September 2002.**

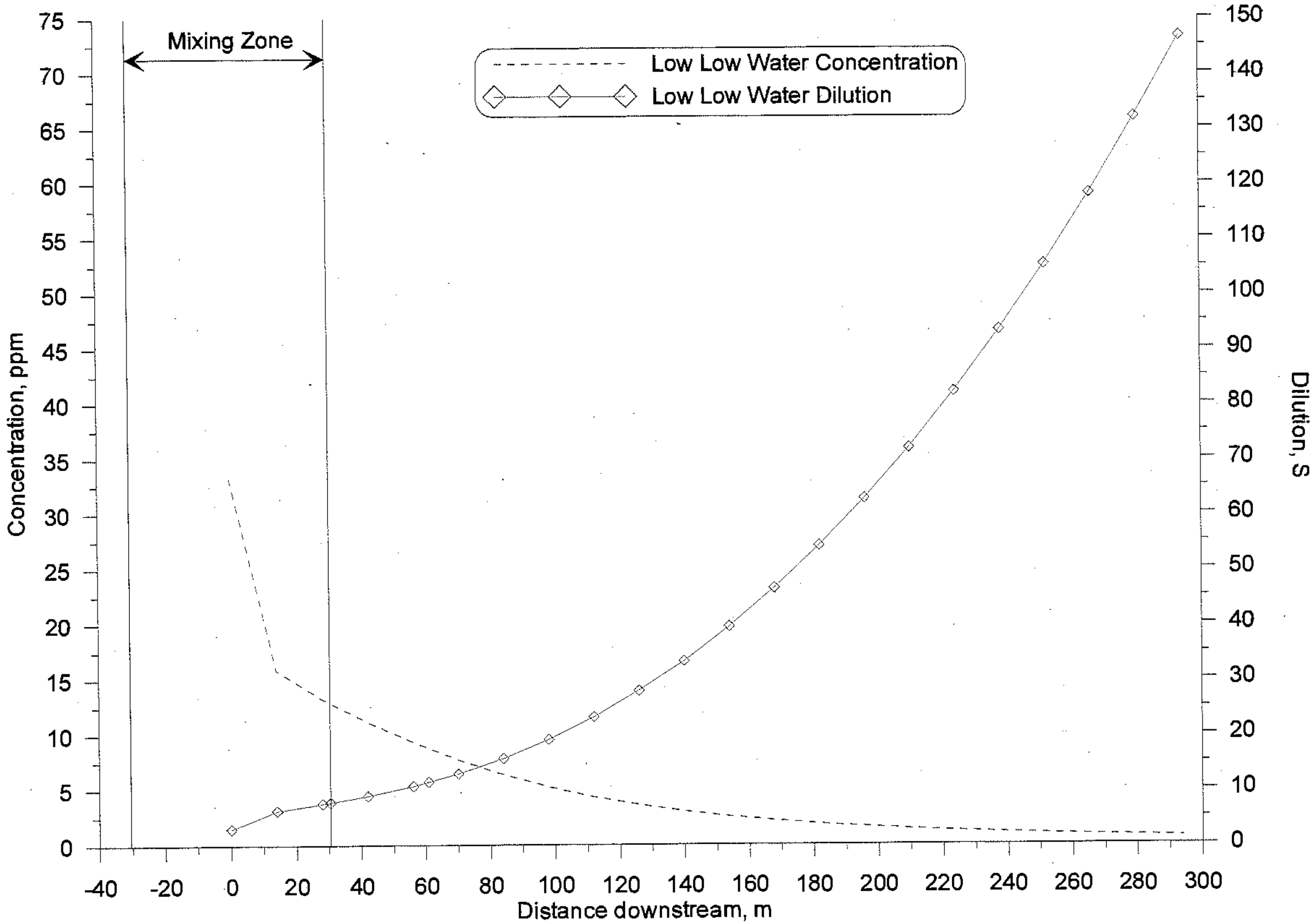


**Figure 62: Dye concentration and dilution for City of Toledo discharge of 0.5 MGD, during low low water slack tide, September 2002.**

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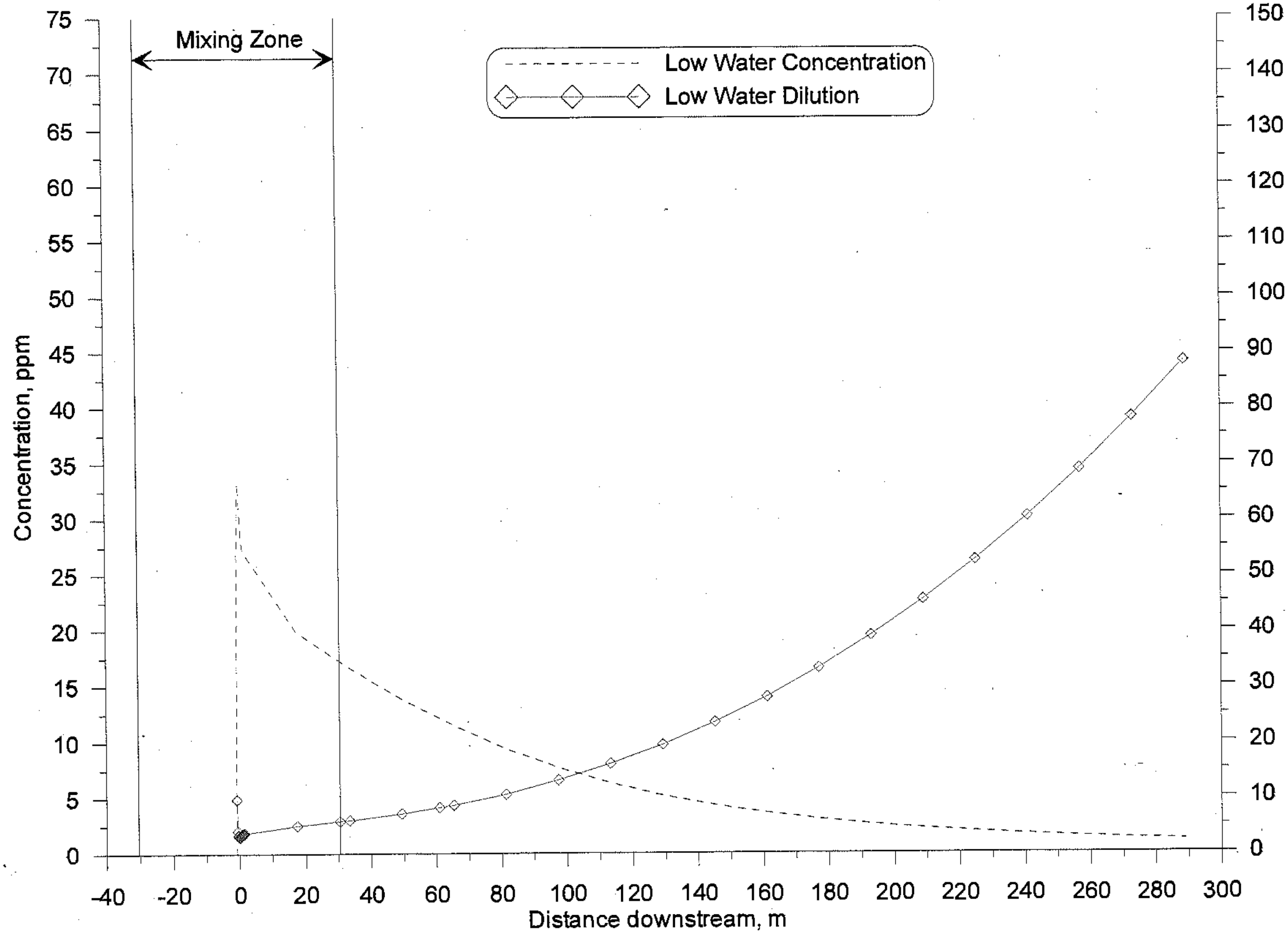

Figure 63: Dye concentration and dilution for City of Toledo discharge of 0.5 MGD, during low low water slack tide, September 2002.

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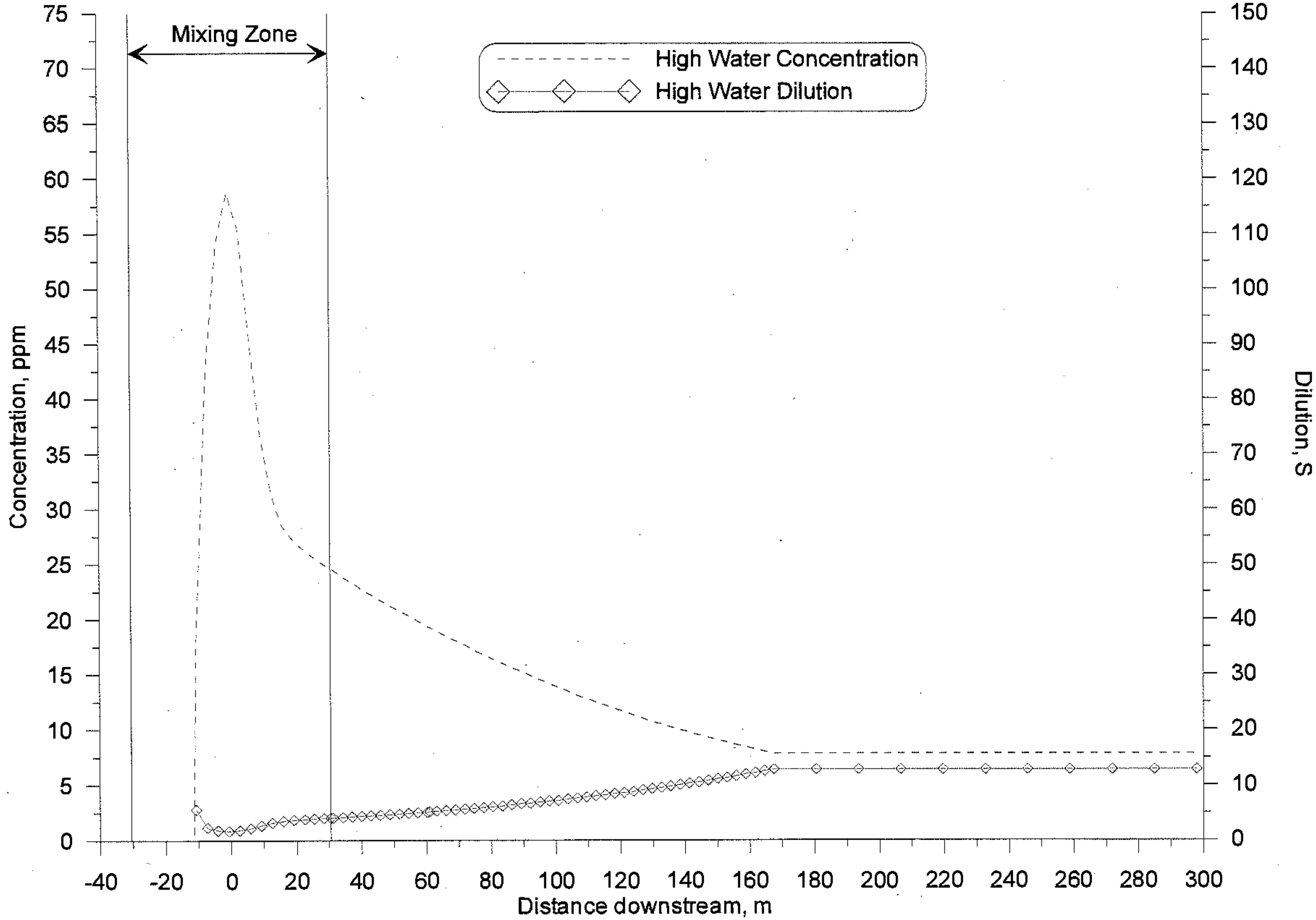


**Figure 64: Dye concentration and dilution for City of Toledo discharge of 0.5 MGD, during high water slack tide, September 2002.**

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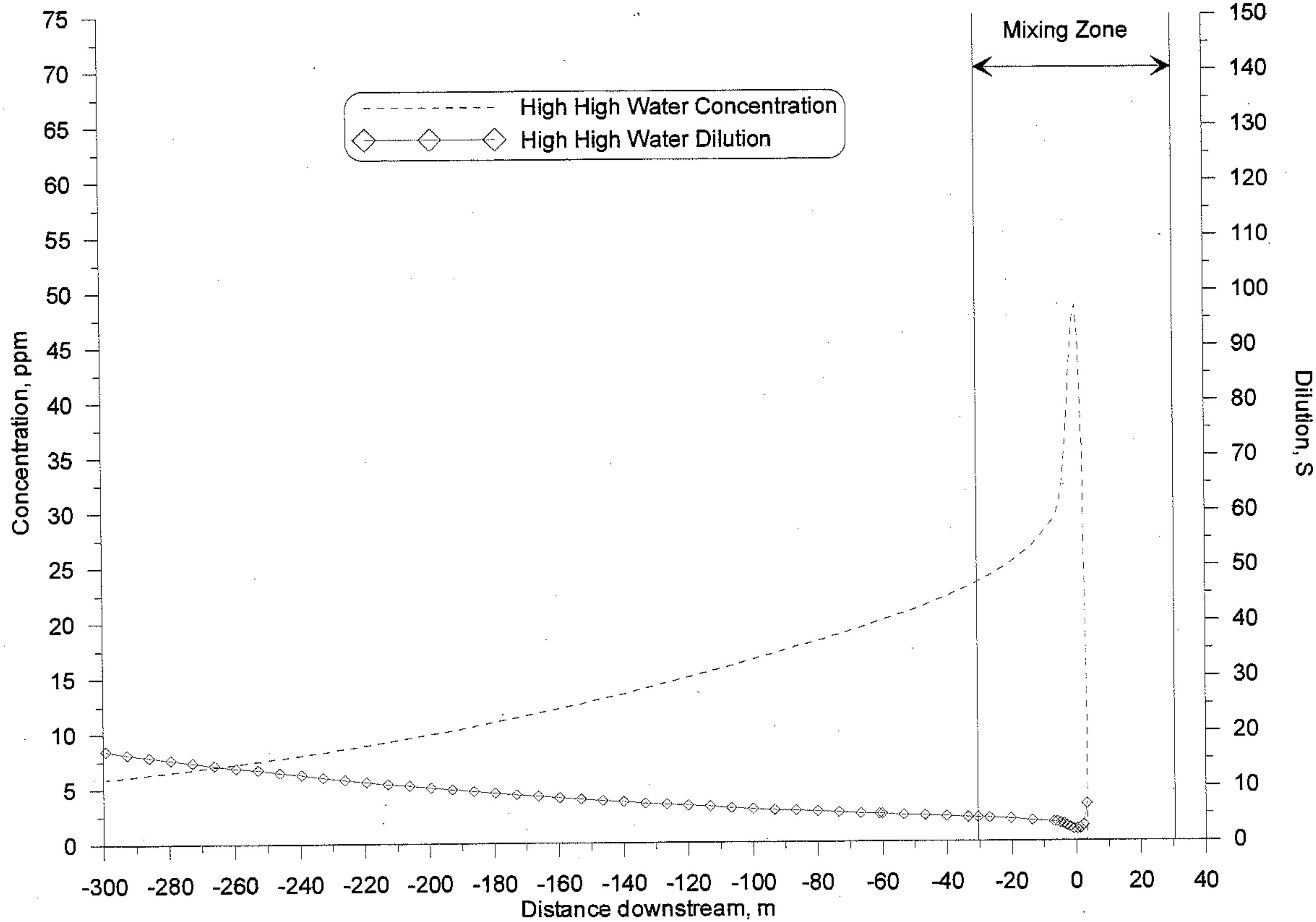


**Figure 65: Dye concentration and dilution for City of Toledo discharge of 0.5 MGD, during high bigh water slack tide, September 2002.**

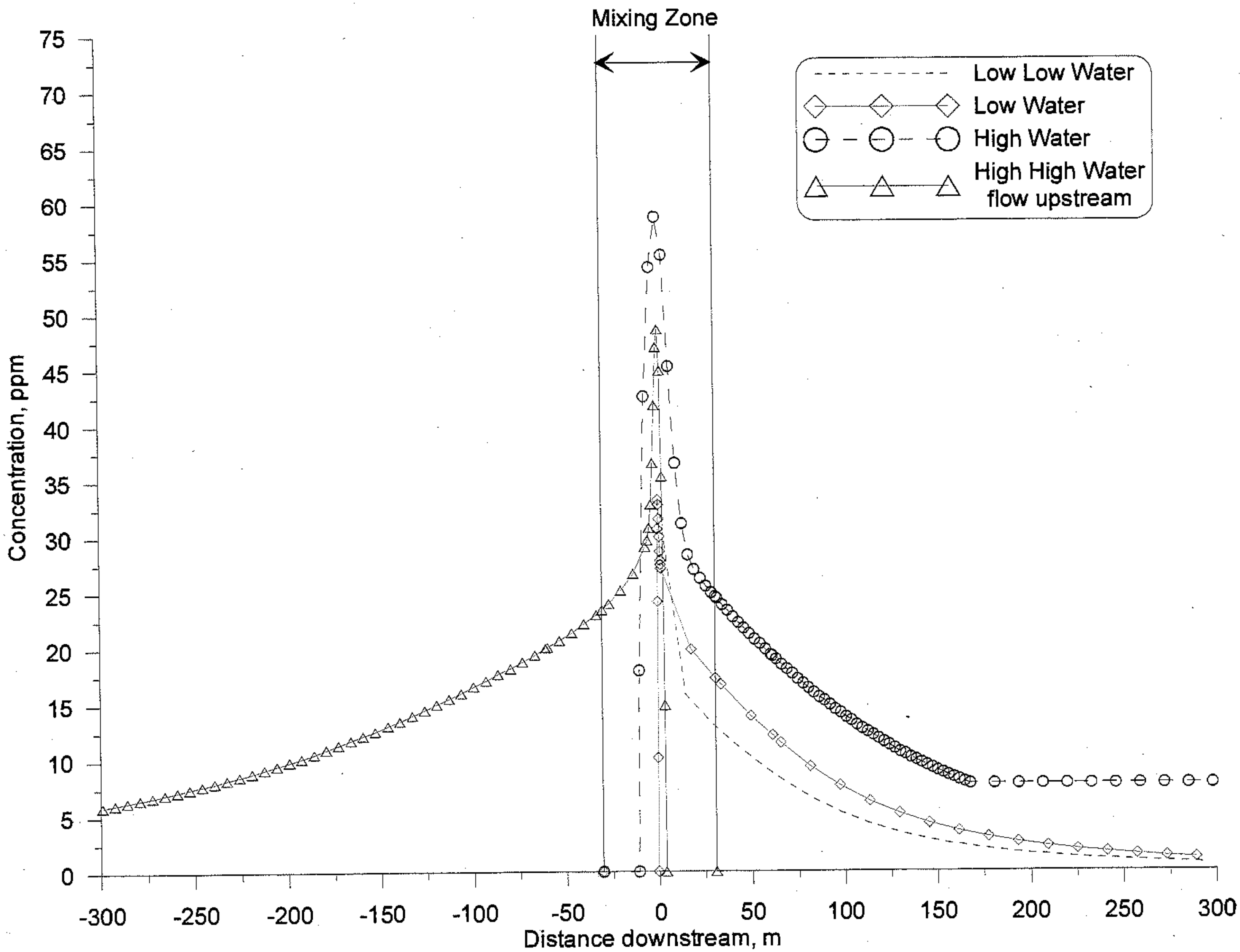


**Figure 66: Dye concentrations for City of Toledo discharge of 0.5 MGD, during various slack tides, September 2002.**

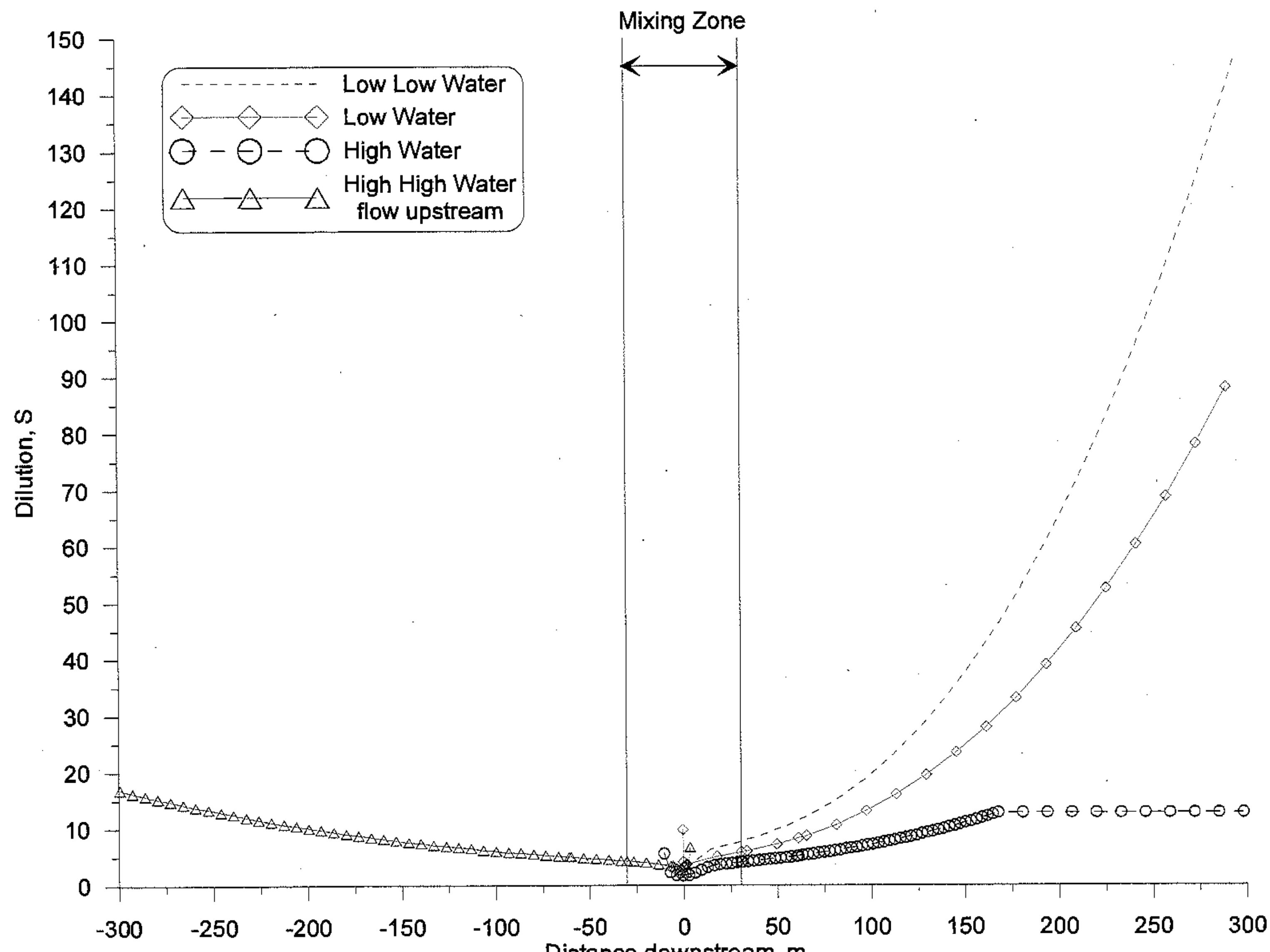


**Figure 67: Dye dilutions for City of Toledo discharge of 0.5 MGD, during various slack tides, September 2002.**



APPENDIX B

# Discussion of Modeling Results

Parameters of interest for the City of Toledo discharge include the following:

- Temperature
- Ammonia
- Chlorine

Each of these parameters is discussed relative to the applicable Oregon DEQ criteria (DEQ, 2004).

## *Temperature*

According to the OAR Chapter 340 Division 041 Figure 220A and B (DEQ, 2004), the Yaquina River in the vicinity of the Toledo outfall is designated as a salmon and trout rearing and migration corridor but not as a designated spawning area. The biologically based numeric criterion is that temperatures may not exceed 18.0°C (60.8°F). The sum of all point sources cannot raise the water by 0.3°C at the point of maximum impact. This would apply to flows at or above the 7Q10 flow.

Hence, in order to measure compliance the 7Q10 flow for September at low low water conditions would be assessed after complete channel mixing to see if the discharge violated the 0.3°C increase in temperature.

In order to calculate this temperature increase, the mixed temperature assuming complete mixing of the WWTP discharge with the Yaquina River at the 7Q10 would be

$$T_{mixed} = \frac{Q_{wwtp}T_{wwtp} + Q_{river}T_{river}}{Q_{wwtp} + Q_{river}}$$

Then the temperature rise above background would be

$$\Delta T = T_{mixed} - T_{river}$$

Table 13 shows the predicted rise above ambient river temperature in case of just mixing the fresh-water inflow with the City of Toledo outflow with the following assumptions:

- River flow is only fresh-water 7Q10 during September
- Assumed 50% frequency of river temperature of river based on sampling data
- Assumed average discharge temperature from WWTP from data from 2004
- Using typical discharge flow rate of 0.5 MGD and assumed wet-weather maximum of 1 MGD (which usually only occurs in wet-winter months)

**Table 13: Prediction of temperature rise above ambient just using 7Q10 for river in September.**

| Scenario | $Q_{river}$, m3/s | $T_{river}$, °C | $Q_{wwtp}$, MGD | $T_{wwtp}$, C | $T_{mixed}$, °C | ΔT, °C |
|---|---|---|---|---|---|---|
| 1 | 0.24 | 18 | 0.5 | 20 | 18.17 | 0.17 |
| 2 | 0.24 | 18 | 1 | 20 | 18.31 | 0.31 |

The predicted temperature rise was below 0.3°C except at the 1 MGD flow. Knowing though that this was overly conservative since the flow at the outfall includes both fresh-water river flow and tidal flow, this analysis was repeated using the flow predicted by the CE-QUAL-W2 model at LLW.

At LLW for the lowest water level during the 7Q10 period, the outflow at the Toledo discharge is brackish and includes tidal flow leaving the river. The flow rate predicted by CE-QUAL-W2 was about 55 $m^3/s$ at this low-water condition at the Butler Bridge. Using these results and the temperature of the river predicted by CE-QUAL-W2, Table 14 shows that the expected temperature rise in the river is well below 0.3°C, and is negligible.

**Table 14: Prediction of temperature rise above ambient CE-QUAL-W2 model predictions for low-low-water for September.**

| Scenario | $Q_{river}$, $m^3/s$ | $T_{river}$, °C | $Q_{wwtp}$, MGD | $T_{wwtp}$, C | $T_{mixed}$, °C | ΔT, °C |
|---|---|---|---|---|---|---|
| 1 | 55.6 | 17.93 | 0.5 | 20 | 17.93 | 0.00 |
| 2 | 55.6 | 17.93 | 1 | 20 | 17.93 | 0.00 |

Another aspect of compliance with the temperature discharge has to do with the temperature mixing zone rules (DEQ, 2005). These rules and how they are met by the City of Toledo are summarized in Table 15.

**Table 15. Temperature mixing zone rules (DEQ, 2005).**

| **Rule: Temperature Thermal Plume Limitations. Temperature mixing zones and effluent limits authorized under 340-041-0028(12)(b) will be established to prevent or minimize the following adverse effects to salmonids inside the mixing zone:** | **City of Toledo compliance** |
|---|---|
| (A) Impairment of an active salmonid spawning area where spawning redds are located or likely to be located. This adverse effect is prevented or minimized by limiting potential fish exposure to temperatures of 13 degrees Celsius (55.4 Fahrenheit) or less for salmon and steelhead, and 9 degrees Celsius (48 degrees Fahrenheit) for bull trout; | Mixing zone is not an active spawning bed |
| (B) Acute impairment or instantaneous lethality is prevented or minimized by limiting potential fish exposure to temperatures of 32.0 degrees Celsius (89.6 degrees Fahrenheit) or more to less than 2 seconds); | Effluent temperatures below 22°C |
| (C) Thermal shock caused by a sudden increase in water temperature is prevented or minimized by limiting potential fish exposure to temperatures of 25.0 degrees Celsius (77.0 degrees Fahrenheit) or more to less than 5 percent of the cross section of 100 percent of the 7Q10 low flow of the water body; the Department may develop additional exposure timing restrictions to prevent thermal shock; and | Effluent temperatures below 22°C |
| (D) Unless the ambient temperature is 21.0 degrees of greater, migration blockage is prevented or minimized by limiting potential fish exposure to temperatures of 21.0 degrees Celsius (69.8 degrees Fahrenheit) or more to less than 25 percent of the cross section of 100 percent of the 7Q10 low flow of the water body. | Current discharge is a surface discharge on the right bank of the river. According to CORMIX the effluent usually becomes bank attached thus minimizing fish exposure during migration. |

### Ammonia

Oregon DEQ (DEQ, 2004) has adopted the freshwater criteria for total ammonia in mg/1 as N in EPA (1999). These criteria are shown below:

Freshwater Acute
The one-hour average concentration of total ammonia nitrogen (in mg N/L) does not exceed, more than once every three years on the average, the CMC (acute criterion) calculated using the following equations. Where salmonid fish are present:

$$CMC = \frac{0.275}{1+10^{7.204-pH}} + \frac{39.0}{1+10^{pH-7.204}}$$

Or where salmonid fish are not present:

$$CMC = \frac{0.411}{1+10^{7.204-pH}} + \frac{58.4}{1+10^{pH-7.204}}$$

Freshwater Chronic
The thirty-day average concentration of total ammonia nitrogen (in mg N/L) does not exceed, more than once every three years on the average, the CCC (chronic criterion) calculated using the following equations. When fish early life stages are present:

$$CCC = \left(\frac{0.0577}{1+10^{7.688-pH}} + \frac{2.487}{1+10^{pH-7.688}}\right) MIN\left(2.85, 1.45X10^{0.028(25-T)}\right)$$

When fish early life stages are absent:

$$CCC = \left(\frac{0.0577}{1+10^{7.688-pH}} + \frac{2.487}{1+10^{pH-7.688}}\right) 1.45X10^{0.028(25-MAX(T,7))}$$

In addition, the highest four-day average within the 30-day period should not exceed 2.5 times the CCC. For the saltwater criterion, values from EPA (1989) are used. A small subset of that information is shown in Table 16 and Table 17 for acute and chronic toxicity criteria, respectively.

**Table 16: Saltwater total ammonia in mg/l as N for criteria maximum concentrations (CMC) or acute criteria.**

| pH | Total Ammonia Concentrations, mg/l as N | | | Salinity, g/kg |
|---|---|---|---|---|
| | 10°C | 15°C | 20°C | |
| 7.0 | 131 | 92 | 62 | 10 |
| 7.4 | 52 | 35 | 25 | 10 |
| 7.8 | 21 | 15 | 10 | 10 |
| 7.0 | 137 | 96 | 64 | 20 |
| 7.4 | 54 | 37 | 27 | 20 |
| 7.8 | 23 | 15 | 11 | 20 |

**Table 17: Saltwater total ammonia in mg/l as N for criteria continuous concentrations (CCC) or chronic criteria.**

| pH | Total Ammonia Concentrations, mg/l as N | | | Salinity, g/kg |
|---|---|---|---|---|
| | 10°C | 15°C | 20°C | |
| 7.0 | 20 | 14 | 9.4 | 10 |
| 7.4 | 7.8 | 5.3 | 3.7 | 10 |
| 7.8 | 3.1 | 2.2 | 1.5 | 10 |
| 7.0 | 20 | 14 | 9.7 | 20 |
| 7.4 | 8.1 | 5.6 | 4.1 | 20 |
| 7.8 | 3.4 | 2.3 | 1.6 | 20 |

During the September 7Q10, the conditions at the outflow are brackish at around 15 g/kg salinity. Hence, the saltwater criterion will be used.

In many cases field data are not available for the receiving water for pH in the vicinity of the outfall. Hence, the following conditions for the receiving water were assumed:

- pH of 7.8 (a conservative value since the river inflow is probably close to 7)
- temperature of the river of 17.93°C (CE-QUAL-W2 result at LLW) at the 7Q10 and critical tidal conditions
- salinity of the river of about 15 g/kg (CE-QUAL-W2 result at LLW) at the 7Q10 and critical tidal conditions

This leads to saltwater acute and chronic toxicity values of 12 mg/l as N and 1.8 mg/l as N, respectively, for total ammonia. (Note that the freshwater criteria would have been 8.1 and 2.5 mg/l as N total ammonia for acute and chronic toxicity at a pH=7.8 and T=17.93°C.) Since the maximum discharged ammonia between January 2004 and March 25, 2005 was 4.8 mg/l as N total ammonia, the current discharge does not violate the acute toxicity value and since the average discharge value was only approximately 0.5 mg/l as N, this is well below the chronic criterion of 1.8 mg/l. Hence, discharge of ammonia at the current levels does not violate toxicity in the Yaquina River.

## *Residual Chlorine*

The DEQ (DEQ, 2004) acute and chronic toxicity standards for chlorine are shown in Table 18.

**Table 18: Chlorine freshwater and saltwater acute and chronic toxicity (DEQ, 2004).**

| Compound | Freshwater Acute Criteria (CMC) | Freshwater Chronic Criteria (CCC) | Saltwater Acute Criteria (CMC) | Saltwater Chronic Criteria (CCC) |
|---|---|---|---|---|
| Chlorine | 19 µg/l | 11 µg/l | 13 µg/l | 7.5 µg/l |

The required dilution to meet chronic toxicity values at the edge of the mixing zone for a given discharge concentration of chlorine are shown in Figure 68 for both freshwater and saltwater criteria.

The City of Toledo discharged an average chlorine residual between January 1, 2004 and March 25, 2005 of 0.79 mg/l, with a range from 0.06 mg/l to 3 mg/l. At the average discharge value of 0.79 mg/l, this would require a dilution of 72 for a freshwater discharge and 106 for a saltwater discharge.

The CORMIX3 model results predicted that the freshwater plume attaches to the bank as the brackish water moves downstream with a dilution after 100 ft of only about 4.8 for a 1 MGD discharge and 7.7 for a 0.5 MGD discharge. Using the simple analytical model assuming a well-mixed vertical inflow into a rectangular channel, the dilution was at a depth of 2.5 m and velocity of about 0.325 m/s (see Figure 55) was about 135 for a discharge of 0.5 MGD.

The CORMIX model is probably a closer representation of the current discharge conditions. Hence, the chronic toxicity standard can be met

- by improving the dilution by about a factor of 10 (this could be achieved by moving the discharge further into the channel, which unfortunately also results in the plume occupying more of the river width and mixing zones cannot occupy the entire river width) or
- by reducing the effluent concentration of chlorine from an average of 0.79 mg/l to about 0.1 mg/l or by using dechlorination or another disinfectant approach (e.g., UV) or
- by discharging during a period of higher dilution

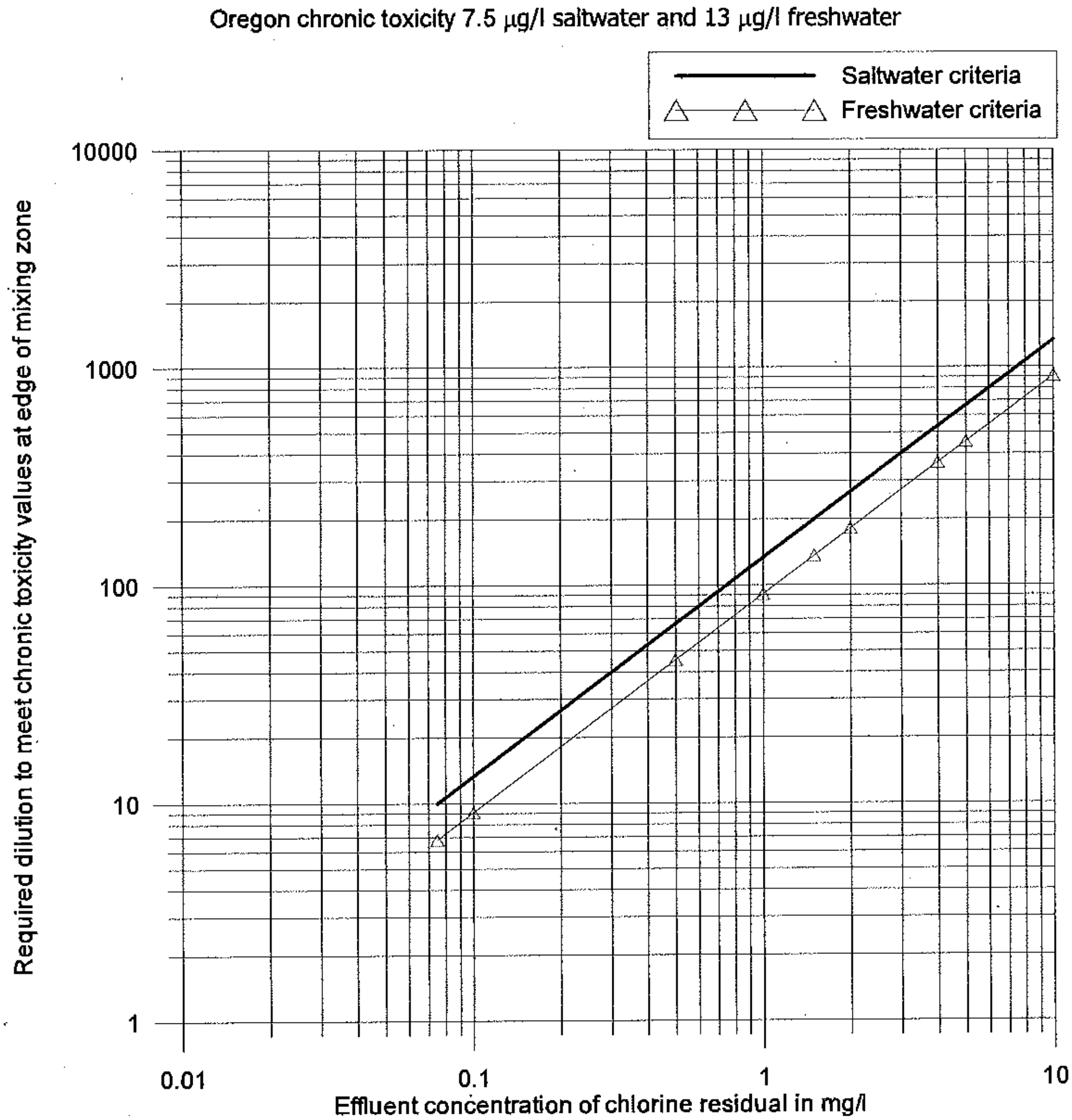


**Figure 68: Required dilution to meet chronic toxicity values at edge of mixing zone for chlorine.**

Is there enough dilution capacity to discharge chlorine as is done at present during the higher discharge months? The CE-QUAL-W2 model was run for the month of May, which has an inflow about 10 times the minimum flow month of September (7Q10 of 2.3 $m^3/s$ for May compared to 0.24 $m^3/s$ for September). Figure 69 shows a frequency distribution of expected velocities during January, May and September 2002. This shows that the mean velocity (representing the fresh water velocity in a tidal average sense) is somewhat larger in May than in September and the extreme velocities are higher for both the flood and the ebb tides. The ebb velocities were still higher in January but the flood velocities were less an account of the higher river flow. In general though, the velocity field for average conditions between the 3 months was only different by about 0.1 m/s. Several additional runs were made with CORMIX to evaluate the impact of different ambient conditions on the required dilution. Several runs were made increasing the velocities done earlier from 0.26 m/s to 2X and 3X that velocity. This was to done to explore the possible impact on mixing for the present outfall configuration of increased velocity to see if one could approach a dilution of 100 at these more extreme flows. Some of these run statistics are summarized in Table 19.

**Table 19. Additional CORMIX simulations evaluating higher flow conditions with a 1 MGD discharge.**

| CORMIX3 Run # | Ambient velocity, m/s | Depth, m | Ambient density, kg/m$^3$ | Width, m | Dilution at edge of mixing zone (100 ft) | Distance, m, downstream to achieve a dilution of 100 |
|---|---|---|---|---|---|---|
| 1 | 0.5 | 3.86 | 1007.86 | 67.5 | 13.4 | 180 |
| 2 | 0.5 | 5 | 1007.86 | 70.0 | 13.3 | 189 |
| 3 | 0.75 | 5 | 1007.86 | 67.5 | 24.5 | 263 |

In all cases, CORMIX predicted a plume that is bound to the right bank limiting its dilution capacity and a dilution prediction under 25. In order to achieve enough mixing for chlorine, dilutions must approach 100. Hence, it is unlikely that within the mandated 100 ft mixing zone, even under higher flow conditions, that the chlorine toxicity numbers can be reached. Other runs could be explored, such as discharge under fresh-water conditions, but since the flow rates are higher in the wetter months (approaching 4 MGD), it is unlikely that enough dilution could be achieved with the current outfall.

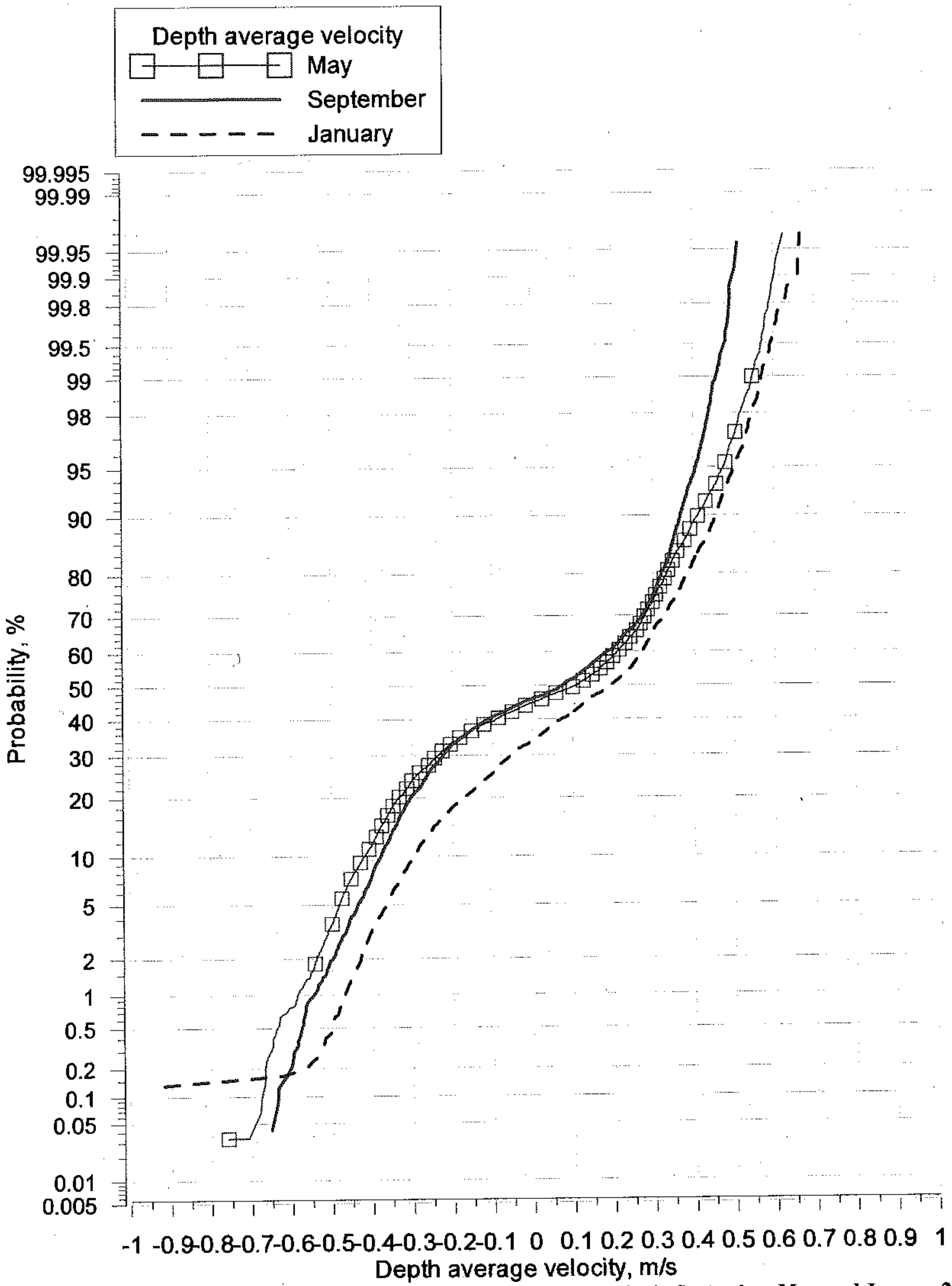


**Figure 69. Comparison of CE-QUAL-W2 predicted depth average velocity in September, May, and January 2002.**

## Summary

This report was prepared to analyze the City of Toledo's mixing of its effluent with the Yaquina River. This is required in the Oregon DEQ permit for the City of Toledo. In order to prepare this assessment the following background information was compiled:

- City of Toledo flow rates
- City of Toledo effluent concentrations of ammonia, residual chlorine, coliform, pH and temperature
- Yaquina River flow rates and statistical assessment of the freshwater 7Q10 and 1 Q10 flow rates (critical freshwater flow conditions for mixing calculations)
- Yaquina River and Bay historical tidal information for assessment of low water level conditions for critical mixing computations
- Resource maps were compiled at Oregon DEQ's request showing shellfish areas, other NPDES permitted discharges, water quality monitoring sites, and beach and water access

In addition statistical analyses were compiled for many of these background data sources in order to assess statistical occurrence and frequency of the parameters of interest in the City of Toledo discharge.

Three different modeling approaches were presented for evaluating the City of Toledo's discharge. These approaches included:

- A CE-QUAL-W2 model of Yaquina River and Bay was developed by EPA (Brown, 2005) and revised by the authors to be a more realistic model of the system. From this model the 7Q10 flows from the Yaquina River at Elk City were used during the month of September 2002 to evaluate critical conditions at the discharge point. From this model, stream velocity, stream depth and width and salinity were obtained for use in the following 2 near-field models.
- An analytical 2-D near field model assuming a rectangular cross-section and vertically well-mixed conditions. This model assumed well-mixed vertical conditions and a neutrally buoyant discharge which probably over-predicted dilution since the discharge was fresher and warmer than the brackish, cool water at a LLW condition. This model predicted dilution factors from 65 to 130 for 1 MGD and 0.5 MGD discharge flow rates.
- The CORMIX3 model for surface discharges was used for the flow parameters derived from the CE-QUAL-W2 model using realistic channel geometry and accounting for the buoyant properties of the surface discharge. The CORMIX model predicted that the plume was an attached plume to the right bank and that little dilution occurred as a result. Predicted dilutions at LLW were between 4.8 and 7.7 for 1 MGD and 0.5 MGD discharge flow rates, respectively.

The parameters of interest for the near-field mixing included:

- Temperature
- Ammonia
- Residual chlorine

In evaluating the current Oregon DEQ criteria for these parameters, only the criteria for chlorine was not in compliance with DEQ standards. Potential options for coming into compliance for chlorine include reducing chlorine to approximately 0.1 mg/l (from a current average of 0.79 mg/l) or to increase the

existing dilution by a factor of more than 10 by an improved diffuser design. The dilution required to achieve compliance was approximately 100. A diffuser design with higher dilution would require the effluent to be distributed over much of the width of the river at low-water conditions and would require an enlargement of the mixing zone beyond the current mixing zone boundaries. Having a diffuser that occupies the entire channel may meet the required dilution for chlorine toxicity, but mixing zones are not allowed to encompass most of the river width. In addition, several simulations were made at times of the year when flows in the Yaquina River were higher than the September low flows. Even for these higher flows there was not enough dilution to come close to a dilution value of 100.

## Recommendations

Since the modeling in this report was based on model results based on field data, there is a need to confirm the predictions in this report by field studies. This would involve verifying the CE-QUAL-W2 and CORMIX model results. The CE-QUAL-W2 results could be verified by comparing field velocity and water level to model predictions. The CORMIX model could be verified by a near-field dye-study release using a conservative tracer.

Areas where further information would be useful:

- Bathymetric data from the Toledo outfall up to Elk City where computations of the tidal prism are important in computing the local hydrodynamics at the City of Toledo outfall
- Acquire additional water level and water quality data collected by the Pacific Coastal Ecology Branch of the U.S. EPA. To be used in refining the CE-QUAL-W2 model of the bay.

Even though the City of Toledo does well in having low effluent coliform bacteria counts, the City of Toledo should explore means to reduce significantly or eliminate effluent concentrations of residual chlorine.

## References

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U.S. Department of Housing and Urban Development (1978) Flood Insurance Study, City of Toledo, Oregon, Lincoln County, Federal Insurance Administration, U.S. Department of Housing and Urban Development.

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River Research and Design, Inc and Pioneer Engineering Corp. (1999) City of Toledo, Oregon, Flood Insurance Map Revision Request, Prepared on behalf of the City of Toledo, Oregon for the Federal Emergency Management Agency.

## Appendix 1 Outfall Structure drawings

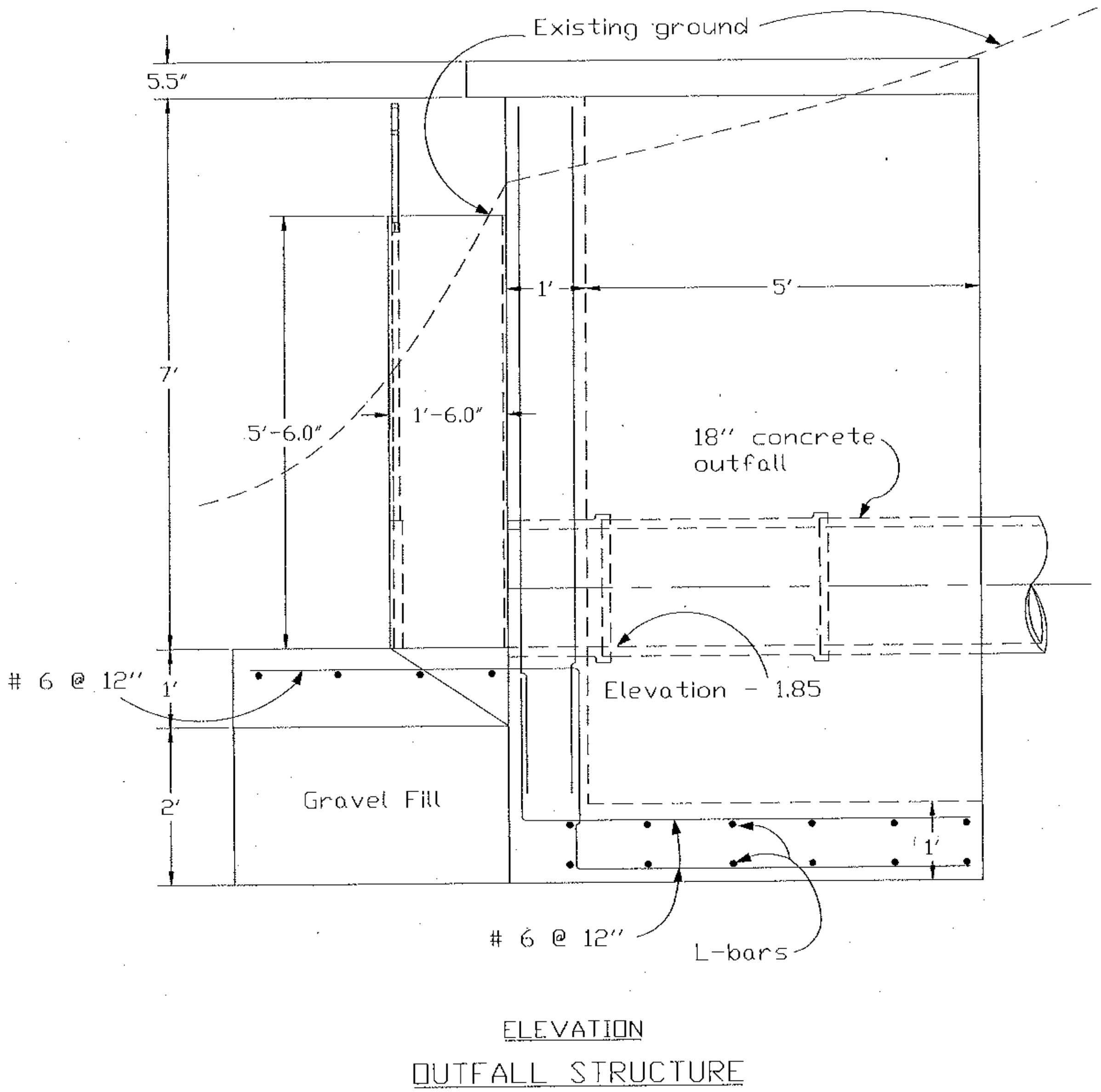


**Figure 70: Outfall Structure Side View - AutoCAD**

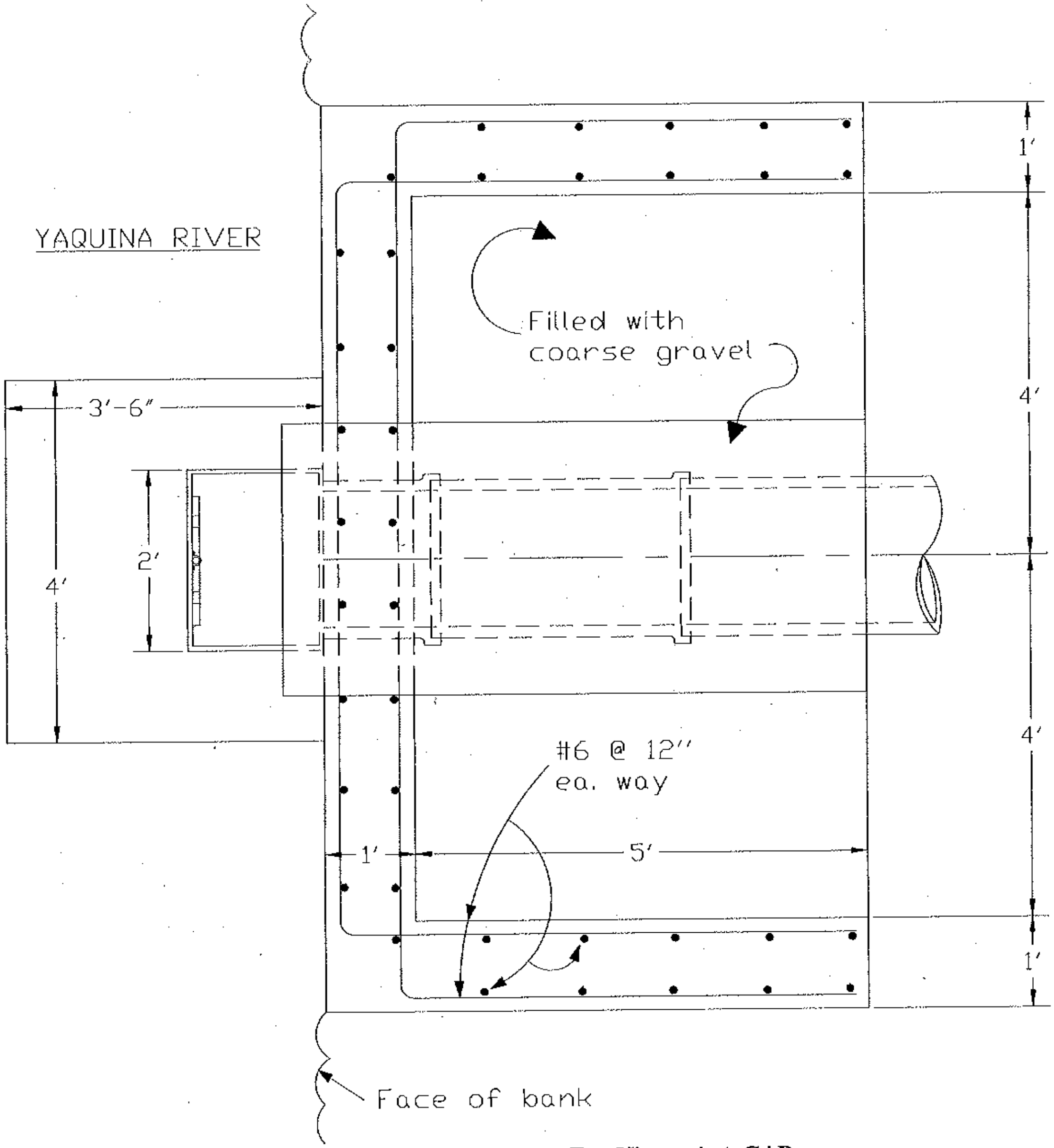


Figure 71: Outfall Structure Top View – AutoCAD

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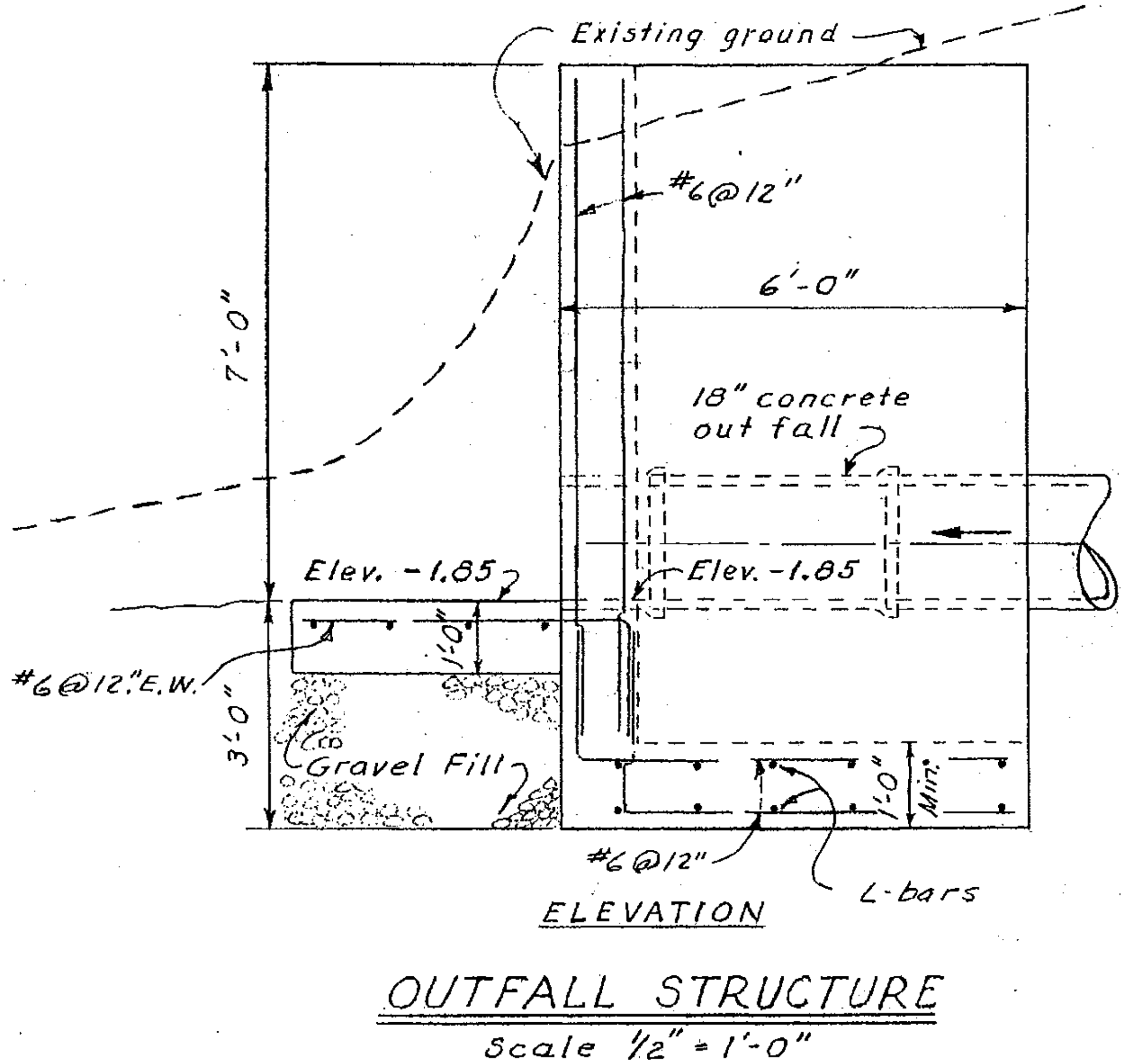


**Figure 72: Outfall Structure Side View - Original**

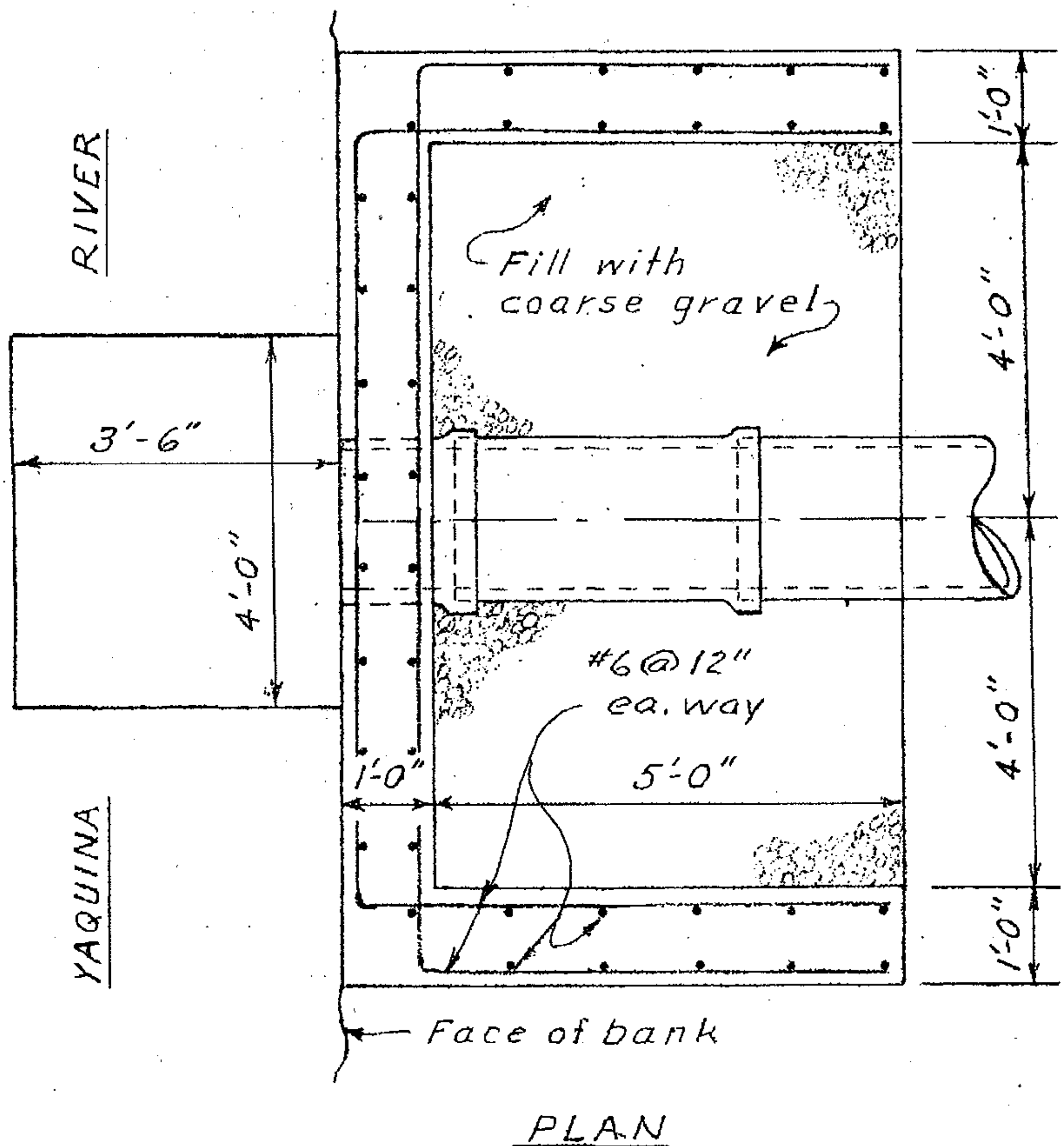


Figure 73: Outfall Structure Top View - Original

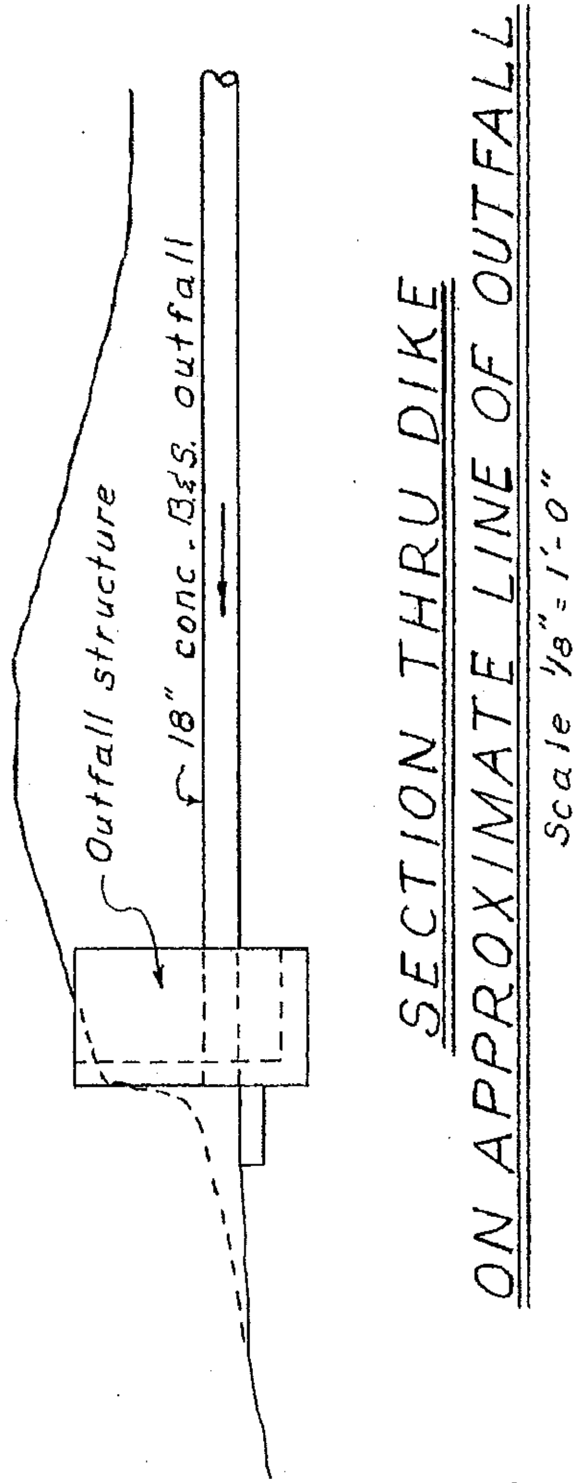


**Figure 74: Outfall Structure and Riverbank**

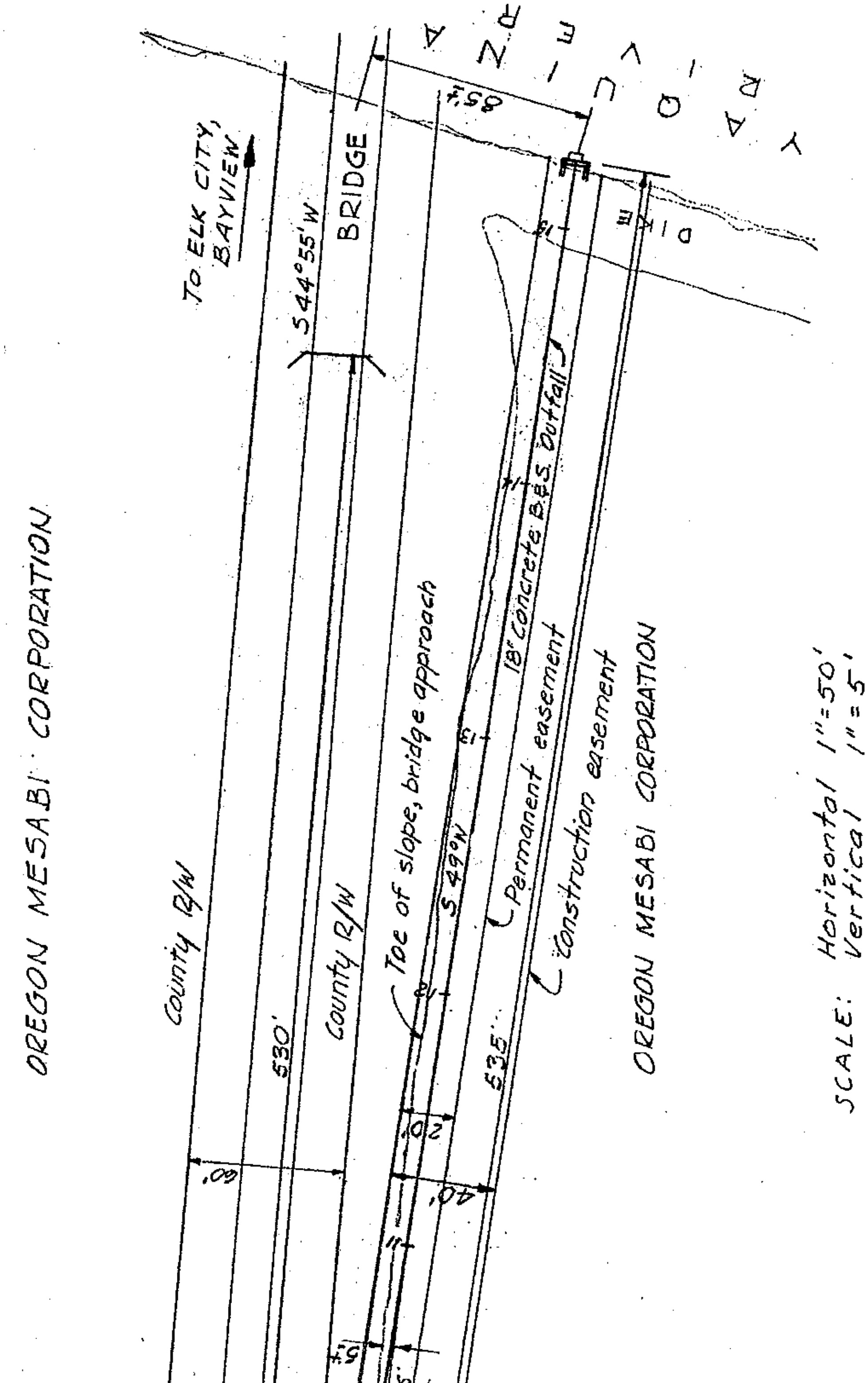


Figure 75: Ariel View of Outfall Piping

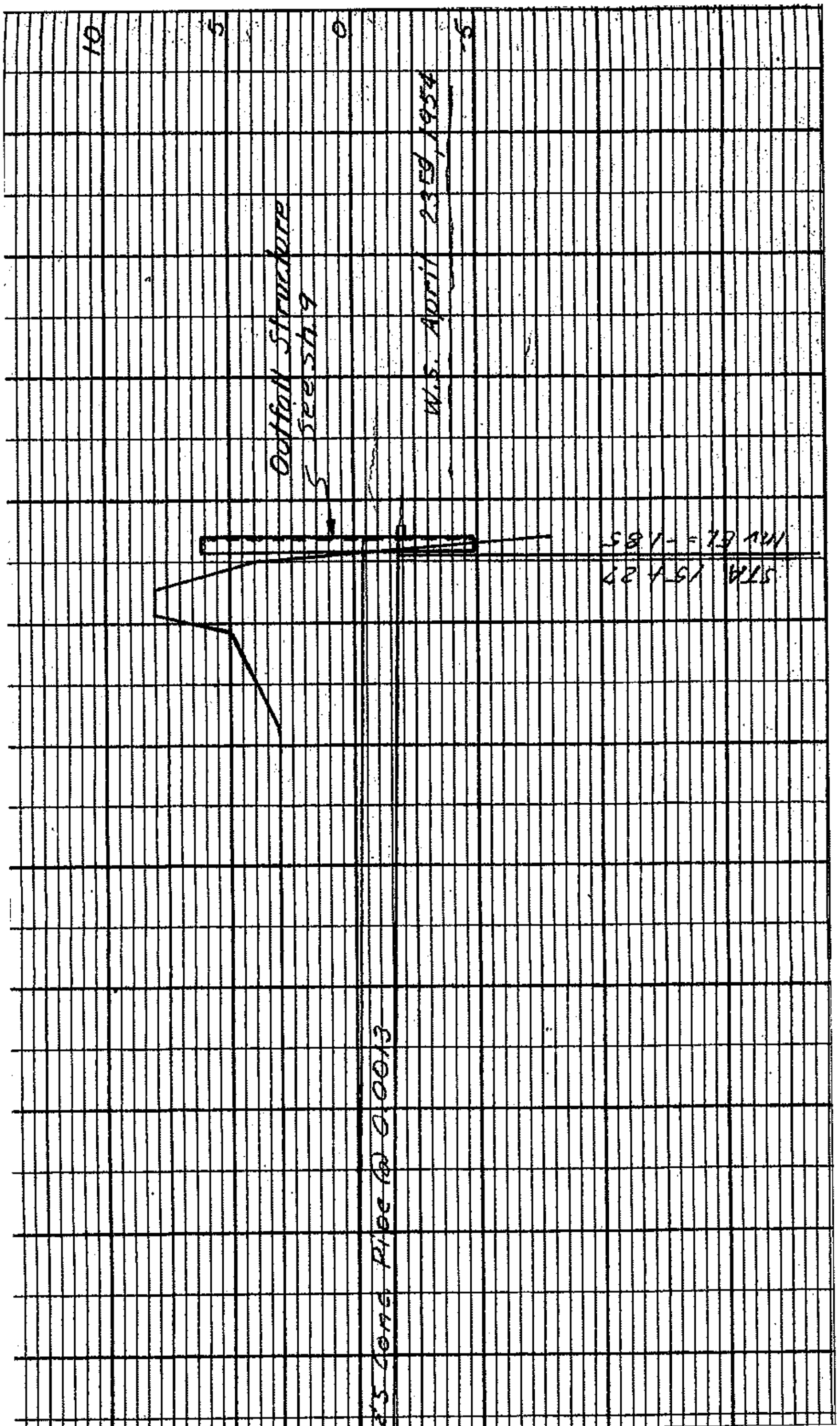


Figure 76: Elevation Diagram of Outfall Piping

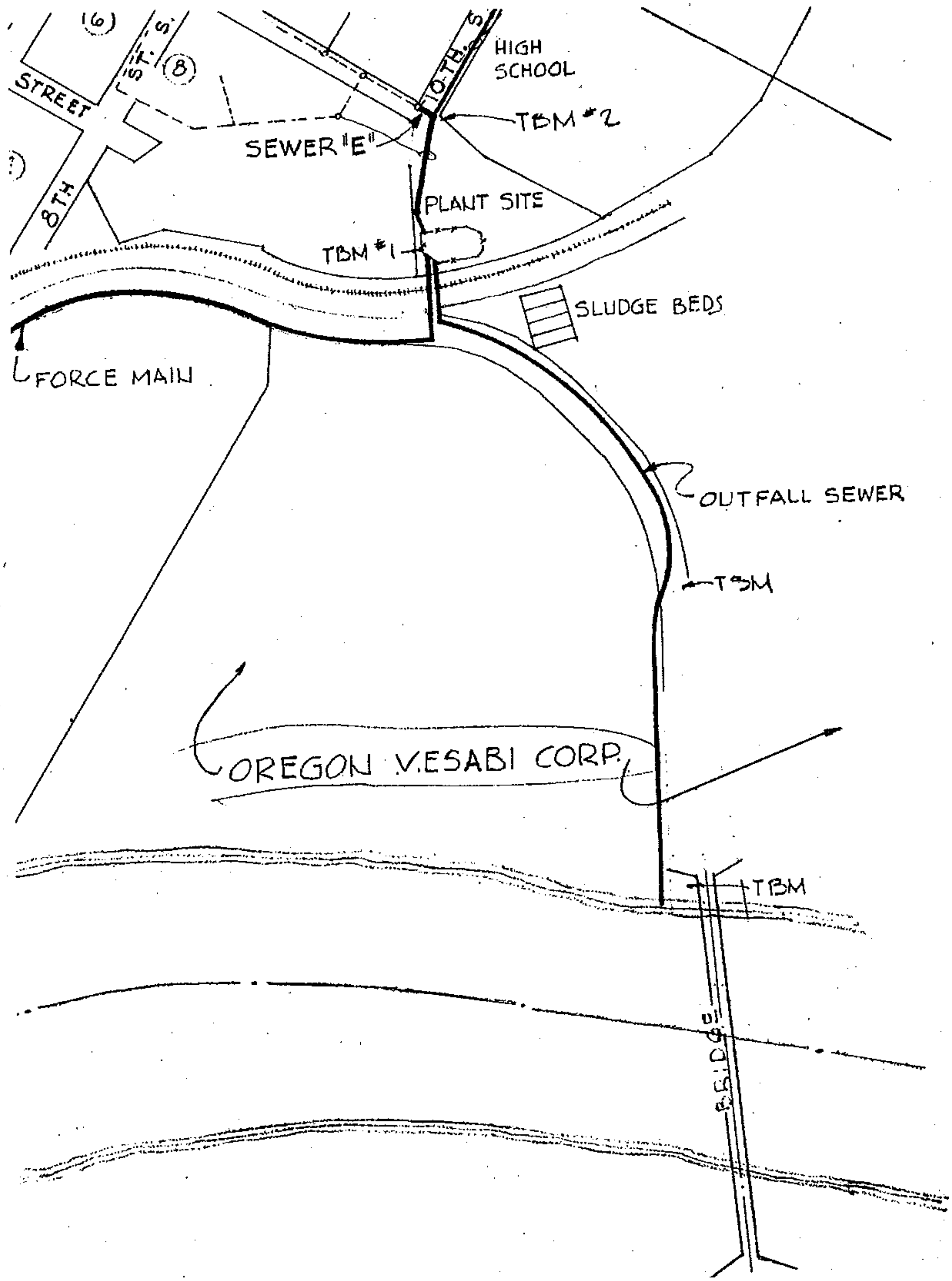


Figure 77: Site Map

## Appendix II – Pictures of Outfall Location

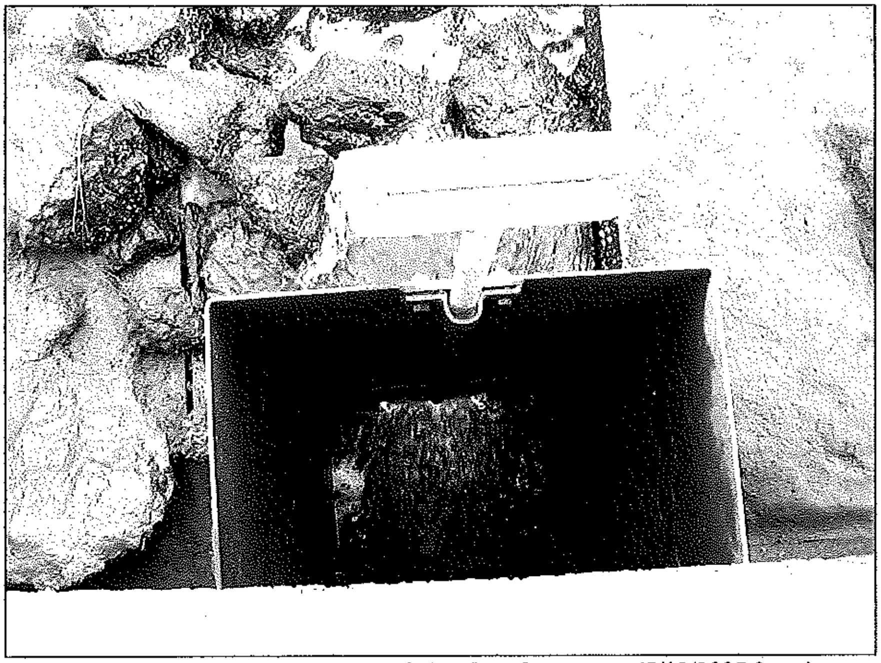

**Figure 78: Looking down at outfall at low, low water (5/13/2005 3 pm).**

**Figure 79: Looking at exposed outfall during low, low water (5/13/2005 3 pm).**

Figure 80: View of outfall from Butler bridge (3/26/2005 11 am).

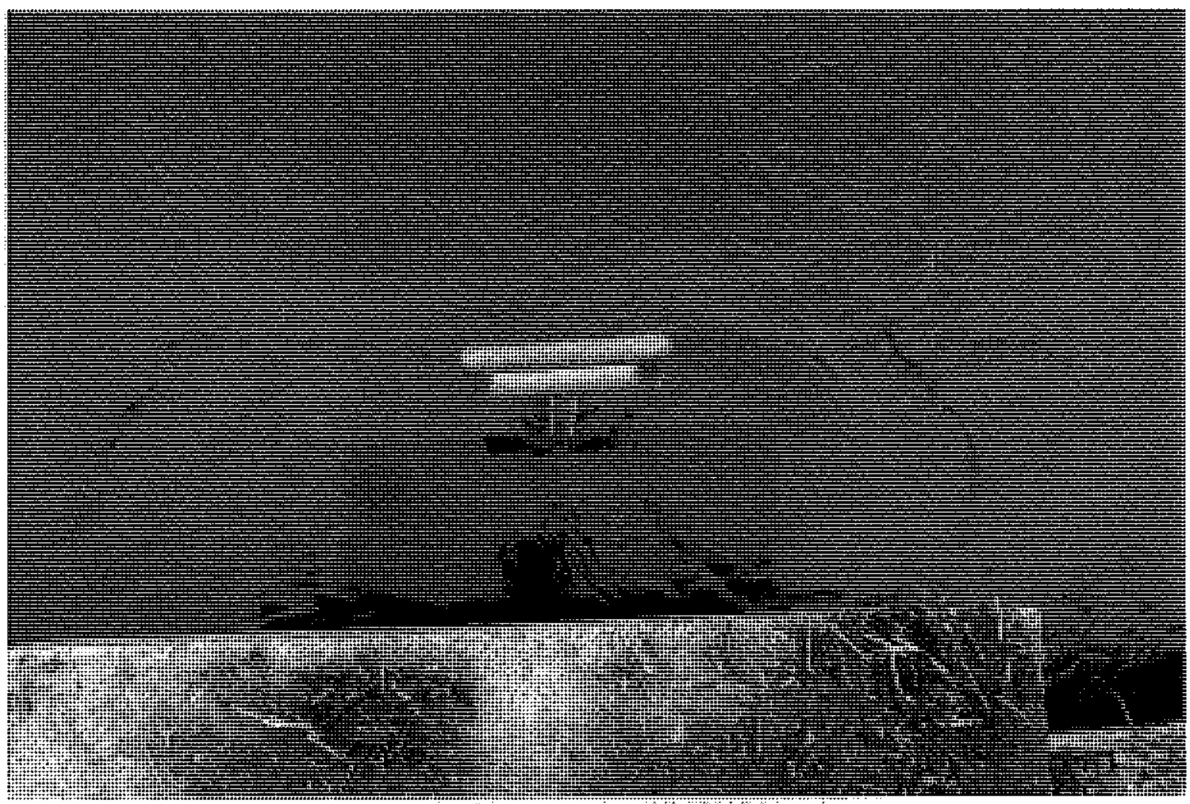

Figure 81: View of outfall (3/26/2005 11 am).

Figure 52: View looking upstream from the Butler Road Bridge (3/26/2005 11 am).

**Figure 83: View looking downstream, outfall in lower right corner (3/26/2005 11 am).**

# Appendix III – CORMIX Model Simulations, 1.0 MGD discharge

## *Low Low Water*

```
CORMIX3 PREDICTION FILE:

                           CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX3:                                              Subsystem version:
 Buoyant Surface Discharges                        CORMIX_v.3.20_____September_1996
-----------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:         City^of^Toledo^Yaquina^River^LLW
 Design case:             LLW3
 FILE NAME:               cormix\sim\LLW3     ,cx3
 Time of Fortran run:     08/26/05--15:24:11

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      67.50  AS     =     260.55  QA     =      55.66  ICHREG= 1
 HA     =       3.86  HD     =       3.86
 UA     =       .214  F      =       .020  USTAR  = .1068E-01
 UW     =      2.400  UWSTAR= .2663E-02
 Uniform density environment
 STRCND=  U           RHOAM  = 1007.8600

DISCHARGE PARAMETERS (metric units)
 BANK   =  RIGHT      DISTB  =        .50  Configuration: protruding_discharge
 SIGMA  =      90.00  HD0    =        .46  SLOPE  =      42.00
 Circular discharge pipe:
 D0     =       .457  A0     =       .164
 Dimensions of equivalent rectangular discharge:
 B0     =       .359  H0     =       .457  A0     = .1640E+00  AR     =      1.273
 U0     =       .267  Q0     =       .044         = .4379E-01
 RHO0   =   998.6407  DRHO0  = .9219E+01  GP0    = .8971E-01
 C0     =  .1000E+03  CUNITS=  ppm
 IPOLL  =  1          KS     = .0000E+00  KD     = .0000E+00

FLUX VARIABLES (metric units)
 Q0     = .4379E-01   M0     = .1169E-01  J0     = .3928E-02
 Associated length scales (meters)
 LQ     =       .40   LM     =       .57  Lm     =        .51  Lb     =        .40

NON-DIMENSIONAL PARAMETERS
 FR0    =      1.40   FRCH   =      1.31  R      =      1.24

FLOW CLASSIFICATION
 333333333333333333333333333333333333333333
 3  Flow class (CORMIX3)         =     PL1     3
 3  Applicable layer depth HS =      3.86      3
 333333333333333333333333333333333333333333

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0     = .1000E+03   CUNITS=  ppm
 NTOX   =  0
 NSTD   =  0
 REGMZ  =  1
 REGSPC=  1           XREG   =     62.00  WREG   =        .00  AREG   =        .00
 XINT   =    700.00   XMAX   =    700.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the WATER SURFACE and at center of discharge
      channel/outlet:        .50 m  from the RIGHT bank/shore.
```

```
     X-axis points downstream
     Y-axis points to left as seen by an observer looking downstream
     Z-axis points vertically upward (in CORMIX3, all values Z = 0.00)
NSTEP = 50 display intervals per module
-----------------------------------------------------------------------------
        TRJBUO     TRJATT     TRJBND     TRJNBY     TRJCOR     DILCOR
C        1.854      1.000       .997       .997      1.849      1.000
-----------------------------------------------------------------------------
BEGIN MOD301: DISCHARGE MODULE

 Efflux conditions:
        X          Y         Z          S         C          BV        BH
       .00        .00      0.00        1.0    .100E+03      .46       .18

END OF MOD301: DISCHARGE MODULE
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD302: ZONE OF FLOW ESTABLISHMENT

  Control volume inflow:
        X          Y         Z          S         C          BV        BH
       .00        .00      0.00        1.0    .100E+03      .46       .18

 VERTICAL MIXING occurs in the initial zone of flow establishment.
 Profile definitions:
   BV = Gaussian 1/e (37%) vertical thickness
   BH = Gaussian 1/e (37%) horizontal half-width, normal to trajectory
   S = hydrodynamic centerline dilution
   C = centerline concentration (includes reaction effects, if any)

  Control volume outflow:
        X          Y         Z          S         C          BV        BH
       .00        .00      0.00        2.8    .363E+02      .46      2.31
 Cumulative travel time =                  0. sec

END OF MOD302: ZONE OF FLOW ESTABLISHMENT
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD331: UPSTREAM INTRUDING PLUME

  Control volume inflow:
        X          Y         Z          S         C          BV        BH
       .00        .00      0.00        2.8    .363E+02      .46      2.31

 UPSTREAM INTRUSION PROPERTIES:
        Upstream intrusion length                  =        1.25 m
        X-position of upstream stagnation point    =       -1.25 m
        Thickness in intrusion region              =         .26 m
        Half-width at downstream end               =        2.58 m
        Thickness at downstream end                =         .26 m

 Profile definitions:
   BV = top-hat thickness,measured vertically
   BH = top-hat half-width, measured horizontally from bank/shoreline
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

        X          Y         Z          S         C          BV        BH
     -1.25        .00      0.00     9999.9    .000E+00      .00       .00
     -1.17        .00      0.00        9.0    .111E+02      .08       .37
      -.81        .00      0.00        3.8    .264E+02      .19       .89
      -.44        .00      0.00        3.0    .335E+02      .24      1.20
      -.07        .00      0.00        2.8    .362E+02      .26      1.45
       .29        .00      0.00        2.8    .359E+02      .26      1.66
       .66        .00      0.00        2.9    .346E+02      .26      1.85
      1.03        .00      0.00        3.0    .330E+02      .26      2.01
      1.39        .00      0.00        3.2    .317E+02      .26      2.17
```

```
      1.76      .00     0.00      3.2  .309E+02   .26      2.32
      2.13      .00     0.00      3.3  .305E+02   .26      2.45
      2.49      .00     0.00      3.3  .302E+02   .26      2.58
 Cumulative travel time =        12. sec

END OF MOD331: UPSTREAM INTRUDING PLUME
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
** End of NEAR-FIELD REGION (NFR) **
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD341: BUOYANT AMBIENT SPREADING

 Plume is ATTACHED to RIGHT bank/shore.
   Plume width is now determined from RIGHT bank/shore.

 Profile definitions:
   BV = top-hat thickness,measured vertically
   BH = top-hat half-width, measured horizontally from bank/shoreline
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

 Plume Stage 2 (bank attached):
       X          Y        Z         S       C        BV       BH
      2.49      -.50     0.00      3.3  .302E+02   .26      2.58
     16.44      -.50     0.00      4.1  .242E+02   .15      5.66
     30.40      -.50     0.00      4.8  .210E+02   .12      8.01
     44.35      -.50     0.00      5.5  .183E+02   .11     10.04
     58.30      -.50     0.00      6.3  .159E+02   .11     11.87
 ** REGULATORY MIXING ZONE BOUNDARY **
 In this prediction interval the plume distance meets or exceeds
   the regulatory value =     62.00 m.
 This is the extent of the REGULATORY MIXING ZONE.
     72.25      -.50     0.00      7.3  .136E+02   .11     13.56
     86.20      -.50     0.00      8.6  .117E+02   .12     15.15
    100.15      -.50     0.00     10.1  .994E+01   .12     16.65
    114.10      -.50     0.00     11.8  .848E+01   .13     18.09
    128.05      -.50     0.00     13.8  .726E+01   .14     19.47
    142.00      -.50     0.00     16.0  .624E+01   .16     20.81
    155.95      -.50     0.00     18.6  .538E+01   .17     22.10
    169.90      -.50     0.00     21.4  .467E+01   .19     23.36
    183.85      -.50     0.00     24.6  .407E+01   .20     24.59
    197.80      -.50     0.00     28.0  .357E+01   .22     25.79
    211.75      -.50     0.00     31.8  .314E+01   .24     26.96
    225.70      -.50     0.00     35.9  .278E+01   .26     28.11
    239.65      -.50     0.00     40.4  .248E+01   .28     29.24
    253.60      -.50     0.00     45.2  .221E+01   .30     30.35
    267.55      -.50     0.00     50.3  .199E+01   .33     31.44
    281.50      -.50     0.00     55.9  .179E+01   .35     32.51
    295.45      -.50     0.00     61.7  .162E+01   .38     33.57
    309.40      -.50     0.00     68.0  .147E+01   .40     34.61
    323.35      -.50     0.00     74.6  .134E+01   .43     35.64
    337.30      -.50     0.00     81.6  .122E+01   .46     36.65
    351.25      -.50     0.00     89.1  .112E+01   .48     37.65
    365.20      -.50     0.00     96.9  .103E+01   .51     38.64
    379.15      -.50     0.00    105.1  .952E+00   .54     39.62
    393.10      -.50     0.00    113.7  .879E+00   .57     40.58
    407.05      -.50     0.00    122.7  .815E+00   .61     41.53
    421.00      -.50     0.00    132.2  .756E+00   .64     42.48
    434.95      -.50     0.00    142.1  .704E+00   .67     43.41
    448.90      -.50     0.00    152.4  .656E+00   .70     44.34
    462.85      -.50     0.00    163.1  .613E+00   .74     45.25
    476.80      -.50     0.00    174.3  .574E+00   .77     46.16
    490.75      -.50     0.00    186.0  .538E+00   .81     47.06
    504.70      -.50     0.00    198.0  .505E+00   .85     47.95
    518.65      -.50     0.00    210.6  .475E+00   .88     48.83
    532.60      -.50     0.00    223.5  .447E+00   .92     49.71
```

```
     546.55      -.50    0.00   237.0  .422E+00    .96     50.58
     560.50      -.50    0.00   250.9  .399E+00   1.00     51.44
     574.45      -.50    0.00   265.3  .377E+00   1.04     52.29
     588.40      -.50    0.00   280.1  .357E+00   1.08     53.14
     602.35      -.50    0.00   295.5  .338E+00   1.12     53.98
     616.30      -.50    0.00   311.3  .321E+00   1.16     54.81
     630.25      -.50    0.00   327.6  .305E+00   1.21     55.64
     644.20      -.50    0.00   344.4  .290E+00   1.25     56.46
     658.15      -.50    0.00   361.7  .276E+00   1.29     57.28
     672.10      -.50    0.00   379.4  .264E+00   1.34     58.09
     686.05      -.50    0.00   397.7  .251E+00   1.38     58.90
     700.00      -.50    0.00   416.5  .240E+00   1.43     59.70
 Cumulative travel time =       3274. sec

 Simulation limit based on maximum specified distance =    700.00 m.
   This is the REGION OF INTEREST limitation.

END OF MOD341: BUOYANT AMBIENT SPREADING
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
CORMIX3: Buoyant Surface Discharges                    End of Prediction File
```

## *Low Water*

```
CORMIX3 PREDICTION FILE:

                      CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX3:                                         Subsystem version:
 Buoyant Surface Discharges               CORMIX_v.3.20_____September_1996
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:          City^of^Toledo^Yaquina^River^LW
 Design case:              LW
 FILE NAME:                cormix\sim\LW1      .cx3
 Time of Fortran run:      08/26/05--15:29:11

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      79.90  AS    =     373.13  QA    =      66.83  ICHREG= 1
 HA     =       4.67  HD    =       4.67
 UA     =       .179  F     =       .019  USTAR = .8679E-02
 UW     =      2.400  UWSTAR= .2663E-02
 Uniform density environment
 STRCND=  U           RHOAM = 1009.0400

DISCHARGE PARAMETERS (metric units)
 BANK   =  RIGHT      DISTB =        .50  Configuration: protruding_discharge
 SIGMA =      90.00  HD0   =        .46  SLOPE =      42.00
 Circular discharge pipe:
 D0     =       .457 A0    =       .164
 Dimensions of equivalent rectangular discharge:
 B0     =       .359 H0    =       .457 A0    = .1640E+00  AR    =       1.273
 U0     =       .267 Q0    =       .044       = .4379E-01
 RHO0   =  998.4258  DRHO0 = .1061E+02  GP0   = .1032E+00
 C0     = .1000E+03  CUNITS=  ppm
 IPOLL =  1          KS    = .0000E+00  KD    = .0000E+00

FLUX VARIABLES (metric units)
 Q0     = .4379E-01  M0    = .1169E-01  J0    = .4517E-02
 Associated length scales (meters)
 LQ     =       .40  LM    =        .53  Lm    =        .60  Lb    =        .79

NON-DIMENSIONAL PARAMETERS
 FR0    =      1.30  FRCH  =       1.22  R     =       1.49
```

```
FLOW CLASSIFICATION
 333333333333333333333333333333333333333333
 3  Flow class (CORMIX3)        =    PL1     3
 3  Applicable layer depth HS =      4.67   3
 333333333333333333333333333333333333333333

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 CO     = .1000E+03  CUNITS=  ppm
 NTOX   =  0
 NSTD   =  0
 REGMZ  =  1
 REGSPC=  1          XREG   =      62.00  WREG   =         .00  AREG   =         .00
 XINT   =      800.00  XMAX   =     800.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the WATER SURFACE and at center of discharge
      channel/outlet:        .50 m  from the RIGHT bank/shore.
    X-axis points downstream
    Y-axis points to left as seen by an observer looking downstream
    Z-axis points vertically upward (in CORMIX3, all values Z = 0.00)
NSTEP = 50 display intervals per module
-----------------------------------------------------------------------------
        TRJBUO     TRJATT     TRJBND     TRJNBY     TRJCOR     DILCOR
C        2.392      1.000       .997       .997      2.385      1.000
-----------------------------------------------------------------------------
BEGIN MOD301: DISCHARGE MODULE

 Efflux conditions:
      X         Y        Z         S         C         BV        BH
     .00       .00     0.00       1.0   .100E+03     .46       .18

END OF MOD301: DISCHARGE MODULE
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD302: ZONE OF FLOW ESTABLISHMENT

  Control volume inflow:
      X         Y        Z         S         C         BV        BH
     .00       .00     0.00       1.0   .100E+03     .46       .18

 VERTICAL MIXING occurs in the initial zone of flow establishment.
 Profile definitions:
   BV = Gaussian 1/e (37%) vertical thickness
   BH = Gaussian 1/e (37%) horizontal half-width, normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

  Control volume outflow:
      X         Y        Z         S         C         BV        BH
     .00       .00     0.00       2.6   .381E+02     .46      2.10
 Cumulative travel time =            0. sec

END OF MOD302: ZONE OF FLOW ESTABLISHMENT
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD331: UPSTREAM INTRUDING PLUME

  Control volume inflow:
      X         Y        Z         S         C         BV        BH
     .00       .00     0.00       2.6   .381E+02     .46      2.10

 UPSTREAM INTRUSION PROPERTIES:
        Upstream intrusion length                   =        1.47 m
        X-position of upstream stagnation point =       -1.47 m
        Thickness in intrusion region               =         .24 m
        Half-width at downstream end                =        3.35 m
```

Thickness at downstream end = .24 m

Profile definitions:
BV = top-hat thickness,measured vertically
BH = top-hat half-width, measured horizontally from bank/shoreline
S = hydrodynamic average (bulk) dilution
C = average (bulk) concentration (includes reaction effects, if any)

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| -1.47 | .00 | 0.00 | 9999.9 | .000E+00 | .00 | .00 |
| -1.38 | .00 | 0.00 | 8.6 | .117E+02 | .07 | .47 |
| -.95 | .00 | 0.00 | 3.6 | .277E+02 | .17 | 1.15 |
| -.52 | .00 | 0.00 | 2.8 | .352E+02 | .22 | 1.56 |
| -.09 | .00 | 0.00 | 2.6 | .380E+02 | .24 | 1.88 |
| .35 | .00 | 0.00 | 2.7 | .376E+02 | .24 | 2.15 |
| .78 | .00 | 0.00 | 2.8 | .360E+02 | .24 | 2.39 |
| 1.21 | .00 | 0.00 | 2.9 | .340E+02 | .24 | 2.61 |
| 1.64 | .00 | 0.00 | 3.1 | .325E+02 | .24 | 2.81 |
| 2.07 | .00 | 0.00 | 3.2 | .315E+02 | .24 | 3.00 |
| 2.51 | .00 | 0.00 | 3.2 | .310E+02 | .24 | 3.18 |
| 2.94 | .00 | 0.00 | 3.3 | .306E+02 | .24 | 3.35 |

Cumulative travel time = 16. sec

END OF MOD331: UPSTREAM INTRUDING PLUME

---

---

** End of NEAR-FIELD REGION (NFR) **

---

---

BEGIN MOD341: BUOYANT AMBIENT SPREADING

Plume is ATTACHED to RIGHT bank/shore.
Plume width is now determined from RIGHT bank/shore.

Profile definitions:
BV = top-hat thickness,measured vertically
BH = top-hat half-width, measured horizontally from bank/shoreline
S = hydrodynamic average (bulk) dilution
C = average (bulk) concentration (includes reaction effects, if any)

Plume Stage 2 (bank attached):

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 2.94 | -.50 | 0.00 | 3.3 | .306E+02 | .24 | 3.35 |
| 18.70 | -.50 | 0.00 | 4.1 | .245E+02 | .13 | 7.55 |
| 34.46 | -.50 | 0.00 | 4.7 | .215E+02 | .11 | 10.74 |
| 50.23 | -.50 | 0.00 | 5.3 | .190E+02 | .10 | 13.47 |

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume distance meets or exceeds
the regulatory value = 62.00 m.
This is the extent of the REGULATORY MIXING ZONE.

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 65.99 | -.50 | 0.00 | 6.0 | .167E+02 | .09 | 15.92 |
| 81.76 | -.50 | 0.00 | 6.9 | .146E+02 | .09 | 18.18 |
| 97.52 | -.50 | 0.00 | 7.9 | .127E+02 | .10 | 20.29 |
| 113.28 | -.50 | 0.00 | 9.1 | .110E+02 | .10 | 22.29 |
| 129.05 | -.50 | 0.00 | 10.5 | .951E+01 | .11 | 24.20 |
| 144.81 | -.50 | 0.00 | 12.1 | .825E+01 | .11 | 26.03 |
| 160.57 | -.50 | 0.00 | 14.0 | .717E+01 | .12 | 27.81 |
| 176.34 | -.50 | 0.00 | 16.0 | .625E+01 | .13 | 29.53 |
| 192.10 | -.50 | 0.00 | 18.3 | .547E+01 | .14 | 31.20 |
| 207.86 | -.50 | 0.00 | 20.8 | .480E+01 | .15 | 32.83 |
| 223.63 | -.50 | 0.00 | 23.6 | .424E+01 | .17 | 34.43 |
| 239.39 | -.50 | 0.00 | 26.6 | .376E+01 | .18 | 35.99 |
| 255.15 | -.50 | 0.00 | 29.9 | .334E+01 | .19 | 37.52 |
| 270.92 | -.50 | 0.00 | 33.5 | .299E+01 | .21 | 39.02 |
| 286.68 | -.50 | 0.00 | 37.3 | .268E+01 | .23 | 40.50 |
| 302.44 | -.50 | 0.00 | 41.5 | .241E+01 | .24 | 41.96 |
| 318.21 | -.50 | 0.00 | 45.9 | .218E+01 | .26 | 43.39 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 333.97 | -.50 | 0.00 | 50.6 | .198E+01 | .28 | 44.80 |
| 349.73 | -.50 | 0.00 | 55.6 | .180E+01 | .29 | 46.19 |
| 365.50 | -.50 | 0.00 | 60.9 | .164E+01 | .31 | 47.56 |
| 381.26 | -.50 | 0.00 | 66.5 | .150E+01 | .33 | 48.92 |
| 397.02 | -.50 | 0.00 | 72.4 | .138E+01 | .35 | 50.26 |
| 412.79 | -.50 | 0.00 | 78.6 | .127E+01 | .37 | 51.58 |
| 428.55 | -.50 | 0.00 | 85.2 | .117E+01 | .39 | 52.89 |
| 444.31 | -.50 | 0.00 | 92.1 | .109E+01 | .42 | 54.18 |
| 460.08 | -.50 | 0.00 | 99.3 | .101E+01 | .44 | 55.46 |
| 475.84 | -.50 | 0.00 | 106.9 | .936E+00 | .46 | 56.73 |
| 491.60 | -.50 | 0.00 | 114.8 | .871E+00 | .48 | 57.98 |
| 507.37 | -.50 | 0.00 | 123.0 | .813E+00 | .51 | 59.23 |
| 523.13 | -.50 | 0.00 | 131.6 | .760E+00 | .53 | 60.46 |
| 538.89 | -.50 | 0.00 | 140.6 | .711E+00 | .56 | 61.67 |
| 554.66 | -.50 | 0.00 | 149.9 | .667E+00 | .58 | 62.88 |
| 570.42 | -.50 | 0.00 | 159.5 | .627E+00 | .61 | 64.08 |
| 586.18 | -.50 | 0.00 | 169.6 | .590E+00 | .63 | 65.27 |
| 601.95 | -.50 | 0.00 | 180.0 | .556E+00 | .66 | 66.44 |
| 617.71 | -.50 | 0.00 | 190.7 | .524E+00 | .69 | 67.61 |
| 633.47 | -.50 | 0.00 | 201.9 | .495E+00 | .72 | 68.77 |
| 649.24 | -.50 | 0.00 | 213.4 | .469E+00 | .75 | 69.92 |
| 665.00 | -.50 | 0.00 | 225.3 | .444E+00 | .77 | 71.06 |
| 680.76 | -.50 | 0.00 | 237.6 | .421E+00 | .80 | 72.20 |
| 696.53 | -.50 | 0.00 | 250.3 | .400E+00 | .83 | 73.32 |
| 712.29 | -.50 | 0.00 | 263.3 | .380E+00 | .86 | 74.44 |
| 728.05 | -.50 | 0.00 | 276.8 | .361E+00 | .90 | 75.55 |
| 743.82 | -.50 | 0.00 | 290.7 | .344E+00 | .93 | 76.65 |
| 759.58 | -.50 | 0.00 | 304.9 | .328E+00 | .96 | 77.74 |
| 775.34 | -.50 | 0.00 | 319.6 | .313E+00 | .99 | 78.83 |
| 791.11 | -.50 | 0.00 | 334.7 | .299E+00 | 1.02 | 79.90 |

Cumulative travel time = 4414. sec
Plume is LATERALLY FULLY MIXED at the end of the buoyant spreading regime.

END OF MOD341: BUOYANT AMBIENT SPREADING

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BEGIN MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = .819E-02 m^2/s
Horizontal diffusivity (initial value) = .102E-01 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to water depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 2 (bank attached):

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 791.11 | -.50 | 0.00 | 334.7 | .299E+00 | 1.02 | 79.90 |
| 791.28 | -.50 | 0.00 | 334.7 | .299E+00 | 1.02 | 79.90 |
| 791.46 | -.50 | 0.00 | 334.7 | .299E+00 | 1.02 | 79.90 |
| 791.64 | -.50 | 0.00 | 334.7 | .299E+00 | 1.02 | 79.90 |
| 791.82 | -.50 | 0.00 | 334.7 | .299E+00 | 1.02 | 79.90 |
| 792.00 | -.50 | 0.00 | 334.7 | .299E+00 | 1.02 | 79.90 |
| 792.17 | -.50 | 0.00 | 334.7 | .299E+00 | 1.02 | 79.90 |
| 792.35 | -.50 | 0.00 | 334.7 | .299E+00 | 1.02 | 79.90 |
| 792.53 | -.50 | 0.00 | 334.8 | .299E+00 | 1.02 | 79.90 |
| 792.71 | -.50 | 0.00 | 334.8 | .299E+00 | 1.02 | 79.90 |
| 792.89 | -.50 | 0.00 | 334.8 | .299E+00 | 1.02 | 79.90 |
| 793.06 | -.50 | 0.00 | 334.8 | .299E+00 | 1.02 | 79.90 |
| 793.24 | -.50 | 0.00 | 334.8 | .299E+00 | 1.02 | 79.90 |
| 793.42 | -.50 | 0.00 | 334.8 | .299E+00 | 1.02 | 79.90 |
| 793.60 | -.50 | 0.00 | 334.8 | .299E+00 | 1.02 | 79.90 |
| 793.77 | -.50 | 0.00 | 334.8 | .299E+00 | 1.02 | 79.90 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 793.95 | -.50 | 0.00 | 334.9 | .299E+00 | 1.02 | 79.90 |
| 794.13 | -.50 | 0.00 | 334.9 | .299E+00 | 1.02 | 79.90 |
| 794.31 | -.50 | 0.00 | 334.9 | .299E+00 | 1.02 | 79.90 |
| 794.49 | -.50 | 0.00 | 334.9 | .299E+00 | 1.02 | 79.90 |
| 794.66 | -.50 | 0.00 | 334.9 | .299E+00 | 1.02 | 79.90 |
| 794.84 | -.50 | 0.00 | 334.9 | .299E+00 | 1.02 | 79.90 |
| 795.02 | -.50 | 0.00 | 334.9 | .299E+00 | 1.02 | 79.90 |
| 795.20 | -.50 | 0.00 | 335.0 | .299E+00 | 1.02 | 79.90 |
| 795.38 | -.50 | 0.00 | 335.0 | .299E+00 | 1.02 | 79.90 |
| 795.55 | -.50 | 0.00 | 335.0 | .299E+00 | 1.02 | 79.90 |
| 795.73 | -.50 | 0.00 | 335.0 | .299E+00 | 1.02 | 79.90 |
| 795.91 | -.50 | 0.00 | 335.0 | .299E+00 | 1.02 | 79.90 |
| 796.09 | -.50 | 0.00 | 335.0 | .298E+00 | 1.02 | 79.90 |
| 796.26 | -.50 | 0.00 | 335.0 | .298E+00 | 1.02 | 79.90 |
| 796.44 | -.50 | 0.00 | 335.0 | .298E+00 | 1.02 | 79.90 |
| 796.62 | -.50 | 0.00 | 335.1 | .298E+00 | 1.02 | 79.90 |
| 796.80 | -.50 | 0.00 | 335.1 | .298E+00 | 1.02 | 79.90 |
| 796.98 | -.50 | 0.00 | 335.1 | .298E+00 | 1.02 | 79.90 |
| 797.15 | -.50 | 0.00 | 335.1 | .298E+00 | 1.02 | 79.90 |
| 797.33 | -.50 | 0.00 | 335.1 | .298E+00 | 1.02 | 79.90 |
| 797.51 | -.50 | 0.00 | 335.1 | .298E+00 | 1.02 | 79.90 |
| 797.69 | -.50 | 0.00 | 335.1 | .298E+00 | 1.02 | 79.90 |
| 797.87 | -.50 | 0.00 | 335.1 | .298E+00 | 1.02 | 79.90 |
| 798.04 | -.50 | 0.00 | 335.2 | .298E+00 | 1.02 | 79.90 |
| 798.22 | -.50 | 0.00 | 335.2 | .298E+00 | 1.02 | 79.90 |
| 798.40 | -.50 | 0.00 | 335.2 | .298E+00 | 1.02 | 79.90 |
| 798.58 | -.50 | 0.00 | 335.2 | .298E+00 | 1.02 | 79.90 |
| 798.75 | -.50 | 0.00 | 335.2 | .298E+00 | 1.02 | 79.90 |
| 798.93 | -.50 | 0.00 | 335.2 | .298E+00 | 1.02 | 79.90 |
| 799.11 | -.50 | 0.00 | 335.2 | .298E+00 | 1.02 | 79.90 |
| 799.29 | -.50 | 0.00 | 335.2 | .298E+00 | 1.03 | 79.90 |
| 799.47 | -.50 | 0.00 | 335.3 | .298E+00 | 1.03 | 79.90 |
| 799.64 | -.50 | 0.00 | 335.3 | .298E+00 | 1.03 | 79.90 |
| 799.82 | -.50 | 0.00 | 335.3 | .298E+00 | 1.03 | 79.90 |
| 800.00 | -.50 | 0.00 | 335.3 | .298E+00 | 1.03 | 79.90 |

```
Cumulative travel time =        4464. sec

Simulation limit based on maximum specified distance =     800.00 m.
  This is the REGION OF INTEREST limitation.

END OF MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
CORMIX3: Buoyant Surface Discharges                    End of Prediction File
```

## *High Water*

```
CORMIX3 PREDICTION FILE:

                      CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX3:                                        Subsystem version:
 Buoyant Surface Discharges                  CORMIX_v.3.20____September_1996
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:          City^of^Toledo^Yaquina^River^HW
 Design case:              HW
 FILE NAME:                cormix\sim\HW1      .cx3
 Time of Fortran run:      08/26/05--15:32:10

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS    =      82.00  AS    =    566.62  QA    =      29.11  ICHREG= 1
 HA    =       6.91  HD    =      6.91
 UA    =       .051  F     =      .016 USTAR = .2332E-02
```

```
 UW     =     3.700 UWSTAR= .4235E-02
 Uniform density environment
 STRCND=  U         RHOAM  = 1015.0000

DISCHARGE PARAMETERS (metric units)
 BANK   =  RIGHT     DISTB =        .50  Configuration: protruding_discharge
 SIGMA =     90.00  HD0    =        .46  SLOPE =      42.00
 Circular discharge pipe:
 D0     =       .457 A0     =        .164
 Dimensions of equivalent rectangular discharge:
 B0     =       .359 H0     =        .457 A0     = .1640E+00  AR     =       1.273
 U0     =       .267 Q0     =        .044        = .4379E-01
 RHO0   =  998.5406  DRHO0 = .1646E+02  GP0    = .1590E+00
 C0     = .1000E+03  CUNITS=  ppm
 IPOLL =  1          KS     = .0000E+00  KD     = .0000E+00

FLUX VARIABLES (metric units)
 Q0     = .4379E-01  M0     = .1169E-01  J0     = .6963E-02
 Associated length scales (meters)
 LQ     =       .40  LM     =        .43  Lm     =      2.10  Lb     =      51.35

NON-DIMENSIONAL PARAMETERS
 FR0    =      1.05  FRCH  =        .99  R      =      5.19

FLOW CLASSIFICATION
 333333333333333333333333333333333333333333
 3  Flow class (CORMIX3)       =      PL1    3
 3  Applicable layer depth HS =      6.91   3
 333333333333333333333333333333333333333333

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0     = .1000E+03  CUNITS=  ppm
 NTOX   =  0
 NSTD   =  0
 REGMZ =  1
 REGSPC=  1          XREG   =     62.00  WREG   =       .00  AREG   =        .00
 XINT   =    820.00  XMAX   =    820.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the WATER SURFACE and at center of discharge
      channel/outlet:         .50 m  from the RIGHT bank/shore.
    X-axis points downstream
    Y-axis points to left as seen by an observer looking downstream
    Z-axis points vertically upward (in CORMIX3, all values Z = 0.00)
NSTEP = 50 display intervals per module
-----------------------------------------------------------------------------

        TRJBUO    TRJATT    TRJBND    TRJNBY    TRJCOR    DILCOR
C        3.880     1.000      .977      .977     3.791     1.000
-----------------------------------------------------------------------------
BEGIN MOD301: DISCHARGE MODULE

 Efflux conditions:
       X         Y        Z        S        C         BV        BH
      .00       .00     0.00      1.0   .100E+03     .46       .18

END OF MOD301: DISCHARGE MODULE
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD302: ZONE OF FLOW ESTABLISHMENT

  Control volume inflow:
       X         Y        Z        S        C         BV        BH
      .00       .00     0.00      1.0   .100E+03     .46       .18

 VERTICAL MIXING occurs in the initial zone of flow establishment.
 Profile definitions:
   BV = Gaussian 1/e (37%) vertical thickness
```

```
   BH = Gaussian 1/e (37%) horizontal half-width, normal to trajectory
   S = hydrodynamic centerline dilution
   C = centerline concentration (includes reaction effects, if any)

  Control volume outflow:
       X          Y         Z         S         C         BV        BH
      .12        .67      0.00       1.6   .638E+02    1.06       .32
 Cumulative travel time =              3. sec

END OF MOD302: ZONE OF FLOW ESTABLISHMENT
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD331: UPSTREAM INTRUDING PLUME

  Control volume inflow:
       X          Y         Z         S         C         BV        BH
      .12        .67      0.00       1.6   .638E+02    1.06       .32

 UPSTREAM INTRUSION PROPERTIES:
        Upstream intrusion length                    =       20.95 m
        X-position of upstream stagnation point      =      -20.95 m
        Thickness in intrusion region                =         .07 m
        Half-width at downstream end                 =       50.67 m
        Thickness at downstream end                  =         .07 m

 Profile definitions:
   BV = top-hat thickness,measured vertically
   BH = top-hat half-width, measured horizontally from bank/shoreline
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

       X          Y         Z         S         C         BV        BH
    -20.95       .00      0.00    9999.9   .000E+00     .00       .00
    -19.69       .00      0.00       5.1   .195E+02     .02      7.17
    -13.53       .00      0.00       2.2   .462E+02     .05     17.41
     -7.37       .00      0.00       1.7   .587E+02     .06     23.55
     -1.22       .00      0.00       1.6   .636E+02     .07     28.39
      4.94       .00      0.00       1.7   .588E+02     .07     32.53
     11.10       .00      0.00       2.2   .457E+02     .07     36.19
     17.26       .00      0.00       2.8   .354E+02     .07     39.51
     23.42       .00      0.00       3.4   .294E+02     .07     42.58
     29.58       .00      0.00       3.8   .264E+02     .07     45.44
     35.74       .00      0.00       4.0   .250E+02     .07     48.13
     41.90       .00      0.00       4.1   .242E+02     .07     50.67
 Cumulative travel time =            816. sec

END OF MOD331: UPSTREAM INTRUDING PLUME
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
** End of NEAR-FIELD REGION (NFR) **
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD341: BUOYANT AMBIENT SPREADING

 Plume is ATTACHED to RIGHT bank/shore.
   Plume width is now determined from RIGHT bank/shore.

 Profile definitions:
   BV = top-hat thickness,measured vertically
   BH = top-hat half-width, measured horizontally from bank/shoreline
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

 Plume Stage 2 (bank attached):
       X          Y         Z         S         C         BV        BH
     41.90      -.50      0.00       4.1   .242E+02     .07     50.63
     43.22      -.50      0.00       4.2   .240E+02     .07     51.40
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```
     44.54      -.50     0.00     4.2  .238E+02     .07     52.16
     45.87      -.50     0.00     4.2  .237E+02     .07     52.91
     47.19      -.50     0.00     4.3  .235E+02     .07     53.65
     48.51      -.50     0.00     4.3  .234E+02     .07     54.38
     49.83      -.50     0.00     4.3  .232E+02     .07     55.11
     51.15      -.50     0.00     4.3  .230E+02     .07     55.83
     52.47      -.50     0.00     4.4  .229E+02     .07     56.54
     53.79      -.50     0.00     4.4  .227E+02     .07     57.24
     55.12      -.50     0.00     4.4  .226E+02     .07     57.94
     56.44      -.50     0.00     4.5  .224E+02     .06     58.63
     57.76      -.50     0.00     4.5  .223E+02     .06     59.32
     59.08      -.50     0.00     4.5  .221E+02     .06     60.00
     60.40      -.50     0.00     4.5  .220E+02     .06     60.67
     61.72      -.50     0.00     4.6  .218E+02     .06     61.34
** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume distance meets or exceeds
  the regulatory value =     62.00 m.
This is the extent of the REGULATORY MIXING ZONE.
     63.04      -.50     0.00     4.6  .217E+02     .06     62.00
     64.36      -.50     0.00     4.6  .216E+02     .06     62.66
     65.69      -.50     0.00     4.7  .214E+02     .06     63.31
     67.01      -.50     0.00     4.7  .213E+02     .06     63.96
     68.33      -.50     0.00     4.7  .211E+02     .06     64.60
     69.65      -.50     0.00     4.8  .210E+02     .06     65.24
     70.97      -.50     0.00     4.8  .208E+02     .06     65.87
     72.29      -.50     0.00     4.8  .207E+02     .06     66.50
     73.61      -.50     0.00     4.9  .206E+02     .06     67.12
     74.94      -.50     0.00     4.9  .204E+02     .06     67.74
     76.26      -.50     0.00     4.9  .203E+02     .06     68.35
     77.58      -.50     0.00     5.0  .202E+02     .06     68.96
     78.90      -.50     0.00     5.0  .200E+02     .06     69.57
     80.22      -.50     0.00     5.0  .199E+02     .06     70.17
     81.54      -.50     0.00     5.1  .197E+02     .06     70.77
     82.86      -.50     0.00     5.1  .196E+02     .06     71.36
     84.18      -.50     0.00     5.1  .195E+02     .06     71.95
     85.51      -.50     0.00     5.2  .193E+02     .06     72.54
     86.83      -.50     0.00     5.2  .192E+02     .06     73.12
     88.15      -.50     0.00     5.2  .191E+02     .06     73.70
     89.47      -.50     0.00     5.3  .190E+02     .06     74.28
     90.79      -.50     0.00     5.3  .188E+02     .06     74.85
     92.11      -.50     0.00     5.3  .187E+02     .06     75.42
     93.43      -.50     0.00     5.4  .186E+02     .06     75.99
     94.76      -.50     0.00     5.4  .184E+02     .06     76.55
     96.08      -.50     0.00     5.5  .183E+02     .06     77.11
     97.40      -.50     0.00     5.5  .182E+02     .06     77.67
     98.72      -.50     0.00     5.5  .181E+02     .06     78.22
    100.04      -.50     0.00     5.6  .179E+02     .06     78.78
    101.36      -.50     0.00     5.6  .178E+02     .06     79.33
    102.68      -.50     0.00     5.7  .177E+02     .06     79.87
    104.01      -.50     0.00     5.7  .176E+02     .06     80.41
    105.33      -.50     0.00     5.7  .174E+02     .06     80.95
    106.65      -.50     0.00     5.8  .173E+02     .06     81.49
    107.97      -.50     0.00     5.8  .172E+02     .06     82.00
Cumulative travel time =        2100. sec
Plume is LATERALLY FULLY MIXED at the end of the buoyant spreading regime.

END OF MOD341: BUOYANT AMBIENT SPREADING
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-----------------------------------------------------------------------------
BEGIN MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

  Vertical diffusivity (initial value)    =  .617E-02 m^2/s
  Horizontal diffusivity (initial value)  =  .771E-02 m^2/s

 Profile definitions:
   BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
      = or equal to water depth, if fully mixed
```

BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 2 (bank attached):

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 107.97 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 122.21 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 136.45 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 150.69 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 164.93 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 179.17 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 193.41 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 207.65 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 221.89 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 236.13 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 250.38 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 264.62 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 278.86 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 293.10 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 307.34 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 321.58 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 335.82 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 350.06 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 364.30 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 378.54 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 392.78 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 407.02 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 421.26 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 435.50 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 449.74 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 463.98 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 478.23 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 492.47 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 506.71 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 520.95 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 535.19 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 549.43 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 563.67 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 577.91 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 592.15 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 606.39 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 620.63 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 634.87 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 649.11 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 663.35 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 677.59 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 691.83 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 706.08 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 720.32 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 734.56 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 748.80 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 763.04 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 777.28 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 791.52 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 805.76 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |
| 820.00 | -.50 | 0.00 | 5.8 | .172E+02 | .06 | 82.00 |

Cumulative travel time = 15938. sec

Simulation limit based on maximum specified distance = 820.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

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CORMIX3: Buoyant Surface Discharges End of Prediction File

## *High High Water*

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CORMIX3 PREDICTION FILE:

                         CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX3:                                         Subsystem version:
 Buoyant Surface Discharges                      CORMIX_v.3.20_____September_1996
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:          City^of^Toledo^Yaquina^River^HHW
 Design case:              HHW
 FILE NAME:                cormix\sim\HHW1     .cx3
 Time of Fortran run:      08/26/05--15:36:10

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =     82.00 AS     =     596.96 QA     =      48.52  ICHREG= 1
 HA     =      7.28 HD     =       7.28
 UA     =      .081 F      =       .016 USTAR = .3658E-02
 UW     =     1.700 UWSTAR= .1854E-02
 Uniform density environment
 STRCND=  U         RHOAM = 1015.8300

DISCHARGE PARAMETERS (metric units)
 BANK  =  RIGHT     DISTB =         .50  Configuration: protruding_discharge
 SIGMA =     90.00  HDO   =         .46  SLOPE =      42.00
 Circular discharge pipe:
 DO    =      .457 AO     =        .164
 Dimensions of equivalent rectangular discharge:
 BO    =      .359 HO     =        .457 AO     = .1640E+00  AR     =      1.273
 UO    =      .267 QO     =        .044        = .4379E-01
 RHOO  =  998.3550 DRHOO = .1748E+02 GPO     = .1687E+00
 CO    = .1000E+03 CUNITS=  ppm
 IPOLL =  1        KS     = .0000E+00 KD     = .0000E+00

FLUX VARIABLES (metric units)
 QO     = .4379E-01  MO    = .1169E-01  JO    = .7387E-02
 Associated length scales (meters)
 LQ     =      .40  LM     =        .41  Lm    =      1.33  Lb    =      13.76

NON-DIMENSIONAL PARAMETERS
 FRO    =     1.02  FRCH   =        .96  R     =      3.28

FLOW CLASSIFICATION
 333333333333333333333333333333333333333333
 3  Flow class (CORMIX3)       =     PL1    3
 3  Applicable layer depth HS =      7.28   3
 333333333333333333333333333333333333333333

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 CO     = .1000E+03  CUNITS=  ppm
 NTOX   =  0
 NSTD   =  0
 REGMZ  =  1
 REGSPC=  1          XREG   =     62.00  WREG   =        .00  AREG   =        .00
 XINT   =    820.00  XMAX   =    820.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the WATER SURFACE and at center of discharge
      channel/outlet:       .50 m  from the RIGHT bank/shore.
    X-axis points downstream
    Y-axis points to left as seen by an observer looking downstream
    Z-axis points vertically upward (in CORMIX3, all values Z = 0.00)
```

```
NSTEP = 50 display intervals per module
--------------------------------------------------------------------------------
        TRJBUO    TRJATT    TRJBND    TRJNBY    TRJCOR    DILCOR
C        3.723     1.000      .991      .991     3.691     1.000
--------------------------------------------------------------------------------
BEGIN MOD301: DISCHARGE MODULE

 Efflux conditions:
      X         Y       Z        S        C         BV        BH
     .00       .00    0.00      1.0   .100E+03     .46       .18

END OF MOD301: DISCHARGE MODULE
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD302: ZONE OF FLOW ESTABLISHMENT

  Control volume inflow:
      X         Y       Z        S        C         BV        BH
     .00       .00    0.00      1.0   .100E+03     .46       .18

 VERTICAL MIXING occurs in the initial zone of flow establishment.
 Profile definitions:
   BV = Gaussian 1/e (37%) vertical thickness
   BH = Gaussian 1/e (37%) horizontal half-width, normal to trajectory
   S = hydrodynamic centerline dilution
   C = centerline concentration (includes reaction effects, if any)

  Control volume outflow:
      X         Y       Z        S        C         BV        BH
     .04       .13    0.00      1.9   .533E+02     .58       .85
 Cumulative travel time =           1. sec

END OF MOD302: ZONE OF FLOW ESTABLISHMENT
--------------------------------------------------------------------------------
--------------------------------------------------------------------------------
BEGIN MOD331: UPSTREAM INTRUDING PLUME

  Control volume inflow:
      X         Y       Z        S        C         BV        BH
     .04       .13    0.00      1.9   .533E+02     .58       .85

 UPSTREAM INTRUSION PROPERTIES:
        Upstream intrusion length                 =        6.72 m
        X-position of upstream stagnation point =       -6.72 m
        Thickness in intrusion region             =         .10 m
        Half-width at downstream end              =       17.42 m
        Thickness at downstream end               =         .10 m

 Profile definitions:
   BV = top-hat thickness,measured vertically
   BH = top-hat half-width, measured horizontally from bank/shoreline
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

      X         Y       Z        S        C         BV        BH
   -6.72       .00    0.00   9999.9   .000E+00     .00       .00
   -6.32       .00    0.00      6.1   .163E+02     .03      2.46
   -4.34       .00    0.00      2.6   .386E+02     .08      5.98
   -2.37       .00    0.00      2.0   .491E+02     .10      8.10
    -.39       .00    0.00      1.9   .531E+02     .10      9.76
    1.59       .00    0.00      2.0   .512E+02     .10     11.18
    3.56       .00    0.00      2.2   .448E+02     .10     12.44
    5.54       .00    0.00      2.6   .385E+02     .10     13.59
    7.51       .00    0.00      2.9   .342E+02     .10     14.64
    9.49       .00    0.00      3.2   .317E+02     .10     15.62
   11.47       .00    0.00      3.3   .306E+02     .10     16.55
   13.44       .00    0.00      3.4   .298E+02     .10     17.42
```

```
 Cumulative travel time =         165. sec

END OF MOD331: UPSTREAM INTRUDING PLUME
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
** End of NEAR-FIELD REGION (NFR) **
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD341: BUOYANT AMBIENT SPREADING

 Plume is ATTACHED to RIGHT bank/shore.
   Plume width is now determined from RIGHT bank/shore.

 Profile definitions:
   BV = top-hat thickness,measured vertically
   BH = top-hat half-width, measured horizontally from bank/shoreline
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

 Plume Stage 2 (bank attached):
        X          Y         Z         S         C         BV        BH
      13.44      -.50      0.00      3.4   .298E+02     .10      17.42
      17.85      -.50      0.00      3.5   .288E+02     .09      19.93
      22.27      -.50      0.00      3.6   .279E+02     .09      22.27
      26.68      -.50      0.00      3.7   .271E+02     .08      24.47
      31.09      -.50      0.00      3.8   .265E+02     .08      26.54
      35.50      -.50      0.00      3.9   .259E+02     .07      28.52
      39.91      -.50      0.00      3.9   .254E+02     .07      30.40
      44.32      -.50      0.00      4.0   .249E+02     .07      32.21
      48.73      -.50      0.00      4.1   .245E+02     .06      33.95
      53.15      -.50      0.00      4.2   .241E+02     .06      35.62
      57.56      -.50      0.00      4.2   .237E+02     .06      37.25
      61.97      -.50      0.00      4.3   .233E+02     .06      38.82
 ** REGULATORY MIXING ZONE BOUNDARY **
 In this prediction interval the plume distance meets or exceeds
   the regulatory value =     62.00 m.
 This is the extent of the REGULATORY MIXING ZONE.
      66.38      -.50      0.00      4.4   .230E+02     .06      40.34
      70.79      -.50      0.00      4.4   .226E+02     .06      41.83
      75.20      -.50      0.00      4.5   .223E+02     .06      43.27
      79.61      -.50      0.00      4.6   .219E+02     .05      44.68
      84.03      -.50      0.00      4.6   .216E+02     .05      46.06
      88.44      -.50      0.00      4.7   .213E+02     .05      47.40
      92.85      -.50      0.00      4.8   .210E+02     .05      48.72
      97.26      -.50      0.00      4.8   .207E+02     .05      50.00
     101.67      -.50      0.00      4.9   .204E+02     .05      51.27
     106.08      -.50      0.00      5.0   .201E+02     .05      52.51
     110.50      -.50      0.00      5.0   .199E+02     .05      53.72
     114.91      -.50      0.00      5.1   .196E+02     .05      54.92
     119.32      -.50      0.00      5.2   .193E+02     .05      56.09
     123.73      -.50      0.00      5.3   .190E+02     .05      57.25
     128.14      -.50      0.00      5.3   .187E+02     .05      58.39
     132.55      -.50      0.00      5.4   .185E+02     .05      59.51
     136.96      -.50      0.00      5.5   .182E+02     .05      60.61
     141.38      -.50      0.00      5.6   .180E+02     .05      61.70
     145.79      -.50      0.00      5.7   .177E+02     .05      62.78
     150.20      -.50      0.00      5.7   .174E+02     .05      63.84
     154.61      -.50      0.00      5.8   .172E+02     .05      64.89
     159.02      -.50      0.00      5.9   .169E+02     .05      65.92
     163.43      -.50      0.00      6.0   .167E+02     .05      66.95
     167.84      -.50      0.00      6.1   .164E+02     .05      67.96
     172.26      -.50      0.00      6.2   .162E+02     .05      68.96
     176.67      -.50      0.00      6.3   .160E+02     .05      69.95
     181.08      -.50      0.00      6.4   .157E+02     .05      70.93
     185.49      -.50      0.00      6.5   .155E+02     .05      71.90
     189.90      -.50      0.00      6.6   .153E+02     .05      72.86
     194.31      -.50      0.00      6.6   .150E+02     .05      73.81
```

| | | | | | | |
|---|---|---|---|---|---|---|
| 198.72 | -.50 | 0.00 | 6.8 | .148E+02 | .05 | 74.76 |
| 203.14 | -.50 | 0.00 | 6.9 | .146E+02 | .05 | 75.69 |
| 207.55 | -.50 | 0.00 | 7.0 | .144E+02 | .05 | 76.62 |
| 211.96 | -.50 | 0.00 | 7.1 | .142E+02 | .05 | 77.54 |
| 216.37 | -.50 | 0.00 | 7.2 | .139E+02 | .05 | 78.45 |
| 220.78 | -.50 | 0.00 | 7.3 | .137E+02 | .05 | 79.35 |
| 225.19 | -.50 | 0.00 | 7.4 | .135E+02 | .05 | 80.25 |
| 229.60 | -.50 | 0.00 | 7.5 | .133E+02 | .05 | 81.14 |
| 234.02 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |

Cumulative travel time = 2877. sec
Plume is LATERALLY FULLY MIXED at the end of the buoyant spreading regime.

END OF MOD341: BUOYANT AMBIENT SPREADING

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BEGIN MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = .555E-02 m^2/s
Horizontal diffusivity (initial value) = .694E-02 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to water depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 2 (bank attached):

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 234.02 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 245.73 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 257.45 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 269.17 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 280.89 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 292.61 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 304.33 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 316.05 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 327.77 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 339.49 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 351.21 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 362.93 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 374.65 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 386.37 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 398.09 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 409.81 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 421.53 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 433.25 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 444.97 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 456.69 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 468.41 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 480.13 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 491.85 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 503.57 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 515.29 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 527.01 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 538.73 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 550.45 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 562.17 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 573.89 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 585.61 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 597.33 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 609.05 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 620.77 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 632.49 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 644.20 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 655.92 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 667.64 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 679.36 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 691.08 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 702.80 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 714.52 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 726.24 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 737.96 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 749.68 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 761.40 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 773.12 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 784.84 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 796.56 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 808.28 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |
| 820.00 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 82.00 |

Cumulative travel time = 10080. sec

Simulation limit based on maximum specified distance = 820.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

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CORMIX3: Buoyant Surface Discharges End of Prediction File

# Appendix IV – Cormix Model Simulations, 0.5 MGD discharge

## *Low Low Water*

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CORMIX3 PREDICTION FILE:

                          CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX3:                                         Subsystem version:
 Buoyant Surface Discharges                       CORMIX_v.3.20_____September_1996
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:          City^of^Toledo^Yaquina^River^LLW
 Design case:              LLW3^0.5^MGD
 FILE NAME:                cormix\sim\LLW305    .cx3
 Time of Fortran run:      08/29/05--14:38:41

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      67.50  AS     =     260.55  QA     =      55.66  ICHREG= 1
 HA     =       3.86  HD     =       3.86
 UA     =       .214  F      =       .020  USTAR  = .1068E-01
 UW     =      2.400  UWSTAR= .2663E-02
 Uniform density environment
 STRCND=  U           RHOAM  = 1007.8600

DISCHARGE PARAMETERS (metric units)
 BANK   =  RIGHT      DISTB  =        .50  Configuration: protruding_discharge
 SIGMA  =      90.00  HD0    =        .46  SLOPE  =      42.00
 Circular discharge pipe:
 D0     =       .457  A0     =       .164
 Dimensions of equivalent rectangular discharge:
 B0     =       .359  H0     =       .457  A0     = .1640E+00  AR     =      1.273
 U0     =       .133  Q0     =       .022         = .2189E-01
 RHO0   =   998.6407  DRHO0  = .9219E+01  GP0    = .8971E-01
 C0     = .1000E+03   CUNITS= ppm
 IPOLL  =  1          KS     = .0000E+00  KD     = .0000E+00

FLUX VARIABLES (metric units)
 Q0     = .2189E-01   M0     = .2923E-02  J0     = .1964E-02
 Associated length scales (meters)
 LQ     =        .40  LM     =        .28  Lm     =        .25  Lb     =        .20

NON-DIMENSIONAL PARAMETERS
 FR0    =        .70  FRCH   =        .65  R      =        .62

FLOW CLASSIFICATION
 333333333333333333333333333333333333333333
 3  Flow class (CORMIX3)       =     PL2     3
 3  Applicable layer depth HS =       3.86   3
 333333333333333333333333333333333333333333

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0     = .1000E+03   CUNITS= ppm
 NTOX   =  0
 NSTD   =  0
 REGMZ  =  1
 REGSPC=  1           XREG   =      62.00  WREG   =        .00  AREG   =        .00
 XINT   =     700.00  XMAX   =     700.00
```

```
X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the WATER SURFACE and at center of discharge
      channel/outlet:        .50 m  from the RIGHT bank/shore.
    X-axis points downstream
    Y-axis points to left as seen by an observer looking downstream
    Z-axis points vertically upward (in CORMIX3, all values Z = 0.00)
NSTEP = 50 display intervals per module
-----------------------------------------------------------------------------
        TRJBUO    TRJATT    TRJBND    TRJNBY    TRJCOR    DILCOR
C        1.854     1.000      .999      .999     1.853     1.000
-----------------------------------------------------------------------------
BEGIN MOD301: DISCHARGE MODULE

 Efflux conditions:
      X         Y        Z        S        C        BV       BH
     .00       .00     0.00      1.0   .100E+03    .46      .18

END OF MOD301: DISCHARGE MODULE
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD302: ZONE OF FLOW ESTABLISHMENT

  Control volume inflow:
      X         Y        Z        S        C        BV       BH
     .00       .00     0.00      1.0   .100E+03    .46      .18

 RAPID DEFLECTION by ambient current:
 Profile definitions:
   BV = top-hat thickness,measured vertically
   BH = top-hat half-width, measured horizontally from bank/shoreline
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

  Control volume outflow:
      X         Y        Z        S        C        BV       BH
     .18       .00     0.00      3.0   .333E+02   1.53      .20
 Cumulative travel time =          1. sec

END OF MOD302: ZONE OF FLOW ESTABLISHMENT
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
** End of NEAR-FIELD REGION (NFR) **
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD341: BUOYANT AMBIENT SPREADING

 Plume is ATTACHED to RIGHT bank/shore.
   Plume width is now determined from RIGHT bank/shore.

 Profile definitions:
   BV = top-hat thickness,measured vertically
   BH = top-hat half-width, measured horizontally from bank/shoreline
   S  = hydrodynamic average (bulk) dilution
   C  = average (bulk) concentration (includes reaction effects, if any)

 Plume Stage 2 (bank attached):
      X         Y        Z        S        C        BV       BH
     .18      -.50     0.00      3.0   .333E+02   1.53      .20
   14.18      -.50     0.00      6.3   .159E+02    .18     3.56
   28.17      -.50     0.00      7.5   .133E+02    .14     5.62
   42.17      -.50     0.00      8.9   .112E+02    .12     7.34
   56.17      -.50     0.00     10.7   .938E+01    .12     8.86
 ** REGULATORY MIXING ZONE BOUNDARY **
 In this prediction interval the plume distance meets or exceeds
   the regulatory value =     62.00 m.
 This is the extent of the REGULATORY MIXING ZONE.
   70.16      -.50     0.00     12.9   .774E+01    .13    10.26
```

| | | | | | | |
|---|---|---|---|---|---|---|
| 84.16 | -.50 | 0.00 | 15.7 | .636E+01 | .14 | 11.57 |
| 98.15 | -.50 | 0.00 | 19.1 | .523E+01 | .15 | 12.81 |
| 112.15 | -.50 | 0.00 | 23.2 | .431E+01 | .17 | 13.99 |
| 126.15 | -.50 | 0.00 | 27.9 | .359E+01 | .19 | 15.12 |
| 140.14 | -.50 | 0.00 | 33.3 | .300E+01 | .21 | 16.22 |
| 154.14 | -.50 | 0.00 | 39.5 | .253E+01 | .23 | 17.28 |
| 168.14 | -.50 | 0.00 | 46.4 | .216E+01 | .26 | 18.30 |
| 182.13 | -.50 | 0.00 | 54.1 | .185E+01 | .29 | 19.31 |
| 196.13 | -.50 | 0.00 | 62.6 | .160E+01 | .32 | 20.28 |
| 210.13 | -.50 | 0.00 | 71.9 | .139E+01 | .35 | 21.24 |
| 224.12 | -.50 | 0.00 | 82.1 | .122E+01 | .38 | 22.17 |
| 238.12 | -.50 | 0.00 | 93.2 | .107E+01 | .41 | 23.09 |
| 252.11 | -.50 | 0.00 | 105.1 | .951E+00 | .45 | 23.99 |
| 266.11 | -.50 | 0.00 | 118.0 | .847E+00 | .49 | 24.87 |
| 280.11 | -.50 | 0.00 | 131.8 | .759E+00 | .52 | 25.74 |
| 294.10 | -.50 | 0.00 | 146.5 | .682E+00 | .56 | 26.59 |
| 308.10 | -.50 | 0.00 | 162.2 | .616E+00 | .61 | 27.43 |
| 322.10 | -.50 | 0.00 | 178.9 | .559E+00 | .65 | 28.26 |
| 336.09 | -.50 | 0.00 | 196.6 | .509E+00 | .69 | 29.08 |
| 350.09 | -.50 | 0.00 | 215.2 | .465E+00 | .74 | 29.89 |
| 364.09 | -.50 | 0.00 | 234.9 | .426E+00 | .78 | 30.68 |
| 378.08 | -.50 | 0.00 | 255.7 | .391E+00 | .83 | 31.47 |
| 392.08 | -.50 | 0.00 | 277.5 | .360E+00 | .88 | 32.25 |
| 406.08 | -.50 | 0.00 | 300.3 | .333E+00 | .93 | 33.01 |
| 420.07 | -.50 | 0.00 | 324.2 | .308E+00 | .98 | 33.77 |
| 434.07 | -.50 | 0.00 | 349.2 | .286E+00 | 1.04 | 34.52 |
| 448.06 | -.50 | 0.00 | 375.4 | .266E+00 | 1.09 | 35.27 |
| 462.06 | -.50 | 0.00 | 402.6 | .248E+00 | 1.15 | 36.00 |
| 476.06 | -.50 | 0.00 | 430.9 | .232E+00 | 1.20 | 36.73 |
| 490.05 | -.50 | 0.00 | 460.4 | .217E+00 | 1.26 | 37.45 |
| 504.05 | -.50 | 0.00 | 491.1 | .204E+00 | 1.32 | 38.17 |
| 518.05 | -.50 | 0.00 | 522.9 | .191E+00 | 1.38 | 38.88 |
| 532.04 | -.50 | 0.00 | 555.8 | .180E+00 | 1.44 | 39.58 |
| 546.04 | -.50 | 0.00 | 590.0 | .169E+00 | 1.50 | 40.28 |
| 560.04 | -.50 | 0.00 | 625.4 | .160E+00 | 1.56 | 40.97 |
| 574.03 | -.50 | 0.00 | 661.9 | .151E+00 | 1.63 | 41.65 |
| 588.03 | -.50 | 0.00 | 699.7 | .143E+00 | 1.69 | 42.33 |
| 602.02 | -.50 | 0.00 | 738.7 | .135E+00 | 1.76 | 43.01 |
| 616.02 | -.50 | 0.00 | 778.9 | .128E+00 | 1.83 | 43.67 |
| 630.02 | -.50 | 0.00 | 820.4 | .122E+00 | 1.90 | 44.34 |
| 644.01 | -.50 | 0.00 | 863.1 | .116E+00 | 1.96 | 45.00 |
| 658.01 | -.50 | 0.00 | 907.1 | .110E+00 | 2.04 | 45.65 |
| 672.01 | -.50 | 0.00 | 952.3 | .105E+00 | 2.11 | 46.30 |
| 686.00 | -.50 | 0.00 | 998.9 | .100E+00 | 2.18 | 46.95 |
| 700.00 | -.50 | 0.00 | 1046.7 | .955E-01 | 2.25 | 47.59 |

Cumulative travel time = 3275. sec

Simulation limit based on maximum specified distance = 700.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD341: BUOYANT AMBIENT SPREADING

---

CORMIX3: Buoyant Surface Discharges End of Prediction File

## *Low Water*

CORMIX3 PREDICTION FILE:

CORNELL MIXING ZONE EXPERT SYSTEM

Subsystem CORMIX3: Subsystem version:
Buoyant Surface Discharges CORMIX_v.3.20____September_1996

---

# APPENDIX B

```
CASE DESCRIPTION
 Site name/label:          City^of^Toledo^Yaquina^River^LW
 Design case:              LW^0.5^MGD
 FILE NAME:                cormix\sim\LW105    .cx3
 Time of Fortran run:      08/29/05--14:40:27

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      79.90  AS     =     373.13  QA     =      66.83  ICHREG= 1
 HA     =       4.67  HD     =       4.67
 UA     =       .179  F      =       .019  USTAR  = .8679E-02
 UW     =      2.400  UWSTAR= .2663E-02
 Uniform density environment
 STRCND=  U           RHOAM  = 1009.0400

DISCHARGE PARAMETERS (metric units)
 BANK   =  RIGHT      DISTB  =        .50  Configuration: protruding_discharge
 SIGMA  =      90.00  HD0    =        .46  SLOPE  =      42.00
 Circular discharge pipe:
 D0     =       .457  A0     =       .164
 Dimensions of equivalent rectangular discharge:
 B0     =       .359  H0     =       .457  A0     = .1640E+00  AR     =      1.273
 U0     =       .133  Q0     =       .022         = .2189E-01
 RHO0   =   998.4258  DRHO0  = .1061E+02  GP0    = .1032E+00
 C0     = .1000E+03   CUNITS=  ppm
 IPOLL  =  1          KS     = .0000E+00  KD     = .0000E+00

FLUX VARIABLES (metric units)
 Q0     = .2189E-01   M0     = .2923E-02  J0     = .2259E-02
 Associated length scales (meters)
 LQ     =        .40  LM     =        .26  Lm     =        .30  Lb     =        .39

NON-DIMENSIONAL PARAMETERS
 FR0    =        .65  FRCH   =        .61  R      =        .74

FLOW CLASSIFICATION
 333333333333333333333333333333333333333333
 3  Flow class (CORMIX3)       =    PL1    3
 3  Applicable layer depth HS  =    4.67   3
 333333333333333333333333333333333333333333

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0     = .1000E+03   CUNITS=  ppm
 NTOX   =  0
 NSTD   =  0
 REGMZ  =  1
 REGSPC=   1          XREG   =      62.00  WREG   =        .00  AREG   =        .00
 XINT   =     800.00  XMAX   =     800.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the WATER SURFACE and at center of discharge
      channel/outlet:      .50 m  from the RIGHT bank/shore.
    X-axis points downstream
    Y-axis points to left as seen by an observer looking downstream
    Z-axis points vertically upward (in CORMIX3, all values Z = 0.00)
NSTEP = 50 display intervals per module
-----------------------------------------------------------------------------
         TRJBUO     TRJATT     TRJBND     TRJNBY     TRJCOR     DILCOR
C         2.392      1.000       .999       .999      2.390      1.000
-----------------------------------------------------------------------------
BEGIN MOD301: DISCHARGE MODULE

 Efflux conditions:
        X          Y          Z          S          C         BV         BH
       .00        .00       0.00        1.0    .100E+03       .46        .18

END OF MOD301: DISCHARGE MODULE
```

---

BEGIN MOD302: ZONE OF FLOW ESTABLISHMENT

Control volume inflow:

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| .00 | .00 | 0.00 | 1.0 | .100E+03 | .46 | .18 |

VERTICAL MIXING occurs in the initial zone of flow establishment.
Profile definitions:
BV = Gaussian 1/e (37%) vertical thickness
BH = Gaussian 1/e (37%) horizontal half-width, normal to trajectory
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Control volume outflow:

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| .00 | .00 | 0.00 | 3.0 | .333E+02 | .46 | 2.75 |

Cumulative travel time = 0. sec

END OF MOD302: ZONE OF FLOW ESTABLISHMENT

---

BEGIN MOD331: UPSTREAM INTRUDING PLUME

Control volume inflow:

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| .00 | .00 | 0.00 | 3.0 | .333E+02 | .46 | 2.75 |

UPSTREAM INTRUSION PROPERTIES:

| | | |
|---|---|---|
| Upstream intrusion length | = | .74 m |
| X-position of upstream stagnation point | = | -.74 m |
| Thickness in intrusion region | = | .25 m |
| Half-width at downstream end | = | 1.80 m |
| Thickness at downstream end | = | .25 m |

Profile definitions:
BV = top-hat thickness,measured vertically
BH = top-hat half-width, measured horizontally from bank/shoreline
S = hydrodynamic average (bulk) dilution
C = average (bulk) concentration (includes reaction effects, if any)

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| -.74 | .00 | 0.00 | 9999.9 | .000E+00 | .00 | .00 |
| -.69 | .00 | 0.00 | 9.8 | .102E+02 | .08 | .25 |
| -.48 | .00 | 0.00 | 4.1 | .242E+02 | .18 | .62 |
| -.26 | .00 | 0.00 | 3.3 | .307E+02 | .23 | .84 |
| -.04 | .00 | 0.00 | 3.0 | .333E+02 | .25 | 1.01 |
| .17 | .00 | 0.00 | 3.0 | .329E+02 | .25 | 1.16 |
| .39 | .00 | 0.00 | 3.2 | .316E+02 | .25 | 1.29 |
| .61 | .00 | 0.00 | 3.3 | .300E+02 | .25 | 1.41 |
| .83 | .00 | 0.00 | 3.5 | .287E+02 | .25 | 1.51 |
| 1.04 | .00 | 0.00 | 3.6 | .279E+02 | .25 | 1.62 |
| 1.26 | .00 | 0.00 | 3.6 | .275E+02 | .25 | 1.71 |
| 1.48 | .00 | 0.00 | 3.7 | .272E+02 | .25 | 1.80 |

Cumulative travel time = 8. sec

END OF MOD331: UPSTREAM INTRUDING PLUME

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** End of NEAR-FIELD REGION (NFR) **

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BEGIN MOD341: BUOYANT AMBIENT SPREADING

Plume is ATTACHED to RIGHT bank/shore.
Plume width is now determined from RIGHT bank/shore.

Profile definitions:
BV = top-hat thickness,measured vertically
BH = top-hat half-width, measured horizontally from bank/shoreline
S = hydrodynamic average (bulk) dilution
C = average (bulk) concentration (includes reaction effects, if any)

Plume Stage 2 (bank attached):

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 1.48 | -.50 | 0.00 | 3.7 | .272E+02 | .25 | 1.80 |
| 17.45 | -.50 | 0.00 | 5.0 | .199E+02 | .11 | 5.51 |
| 33.42 | -.50 | 0.00 | 6.0 | .167E+02 | .09 | 8.16 |
| 49.39 | -.50 | 0.00 | 7.2 | .139E+02 | .08 | 10.40 |

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume distance meets or exceeds
the regulatory value = 62.00 m.
This is the extent of the REGULATORY MIXING ZONE.

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 65.36 | -.50 | 0.00 | 8.7 | .115E+02 | .09 | 12.41 |
| 81.33 | -.50 | 0.00 | 10.6 | .939E+01 | .09 | 14.26 |
| 97.30 | -.50 | 0.00 | 13.1 | .766E+01 | .10 | 15.99 |
| 113.27 | -.50 | 0.00 | 16.0 | .627E+01 | .11 | 17.63 |
| 129.24 | -.50 | 0.00 | 19.4 | .516E+01 | .12 | 19.20 |
| 145.21 | -.50 | 0.00 | 23.4 | .428E+01 | .14 | 20.71 |
| 161.18 | -.50 | 0.00 | 27.9 | .358E+01 | .15 | 22.17 |
| 177.15 | -.50 | 0.00 | 33.1 | .302E+01 | .17 | 23.58 |
| 193.12 | -.50 | 0.00 | 38.9 | .257E+01 | .19 | 24.96 |
| 209.09 | -.50 | 0.00 | 45.4 | .220E+01 | .21 | 26.30 |
| 225.06 | -.50 | 0.00 | 52.5 | .190E+01 | .23 | 27.61 |
| 241.03 | -.50 | 0.00 | 60.3 | .166E+01 | .26 | 28.89 |
| 257.01 | -.50 | 0.00 | 68.8 | .145E+01 | .28 | 30.15 |
| 272.98 | -.50 | 0.00 | 78.1 | .128E+01 | .30 | 31.38 |
| 288.95 | -.50 | 0.00 | 88.1 | .114E+01 | .33 | 32.60 |
| 304.92 | -.50 | 0.00 | 98.8 | .101E+01 | .36 | 33.79 |
| 320.89 | -.50 | 0.00 | 110.3 | .907E+00 | .39 | 34.96 |
| 336.86 | -.50 | 0.00 | 122.6 | .816E+00 | .41 | 36.11 |
| 352.83 | -.50 | 0.00 | 135.7 | .737E+00 | .45 | 37.25 |
| 368.80 | -.50 | 0.00 | 149.6 | .669E+00 | .48 | 38.37 |
| 384.77 | -.50 | 0.00 | 164.3 | .609E+00 | .51 | 39.47 |
| 400.74 | -.50 | 0.00 | 179.8 | .556E+00 | .54 | 40.57 |
| 416.71 | -.50 | 0.00 | 196.2 | .510E+00 | .58 | 41.64 |
| 432.68 | -.50 | 0.00 | 213.5 | .468E+00 | .61 | 42.71 |
| 448.65 | -.50 | 0.00 | 231.6 | .432E+00 | .65 | 43.76 |
| 464.62 | -.50 | 0.00 | 250.6 | .399E+00 | .68 | 44.80 |
| 480.59 | -.50 | 0.00 | 270.6 | .370E+00 | .72 | 45.83 |
| 496.56 | -.50 | 0.00 | 291.4 | .343E+00 | .76 | 46.85 |
| 512.53 | -.50 | 0.00 | 313.1 | .319E+00 | .80 | 47.86 |
| 528.50 | -.50 | 0.00 | 335.8 | .298E+00 | .84 | 48.86 |
| 544.47 | -.50 | 0.00 | 359.4 | .278E+00 | .88 | 49.84 |
| 560.44 | -.50 | 0.00 | 383.9 | .260E+00 | .92 | 50.82 |
| 576.41 | -.50 | 0.00 | 409.4 | .244E+00 | .97 | 51.79 |
| 592.38 | -.50 | 0.00 | 435.9 | .229E+00 | 1.01 | 52.76 |
| 608.35 | -.50 | 0.00 | 463.3 | .216E+00 | 1.05 | 53.71 |
| 624.33 | -.50 | 0.00 | 491.7 | .203E+00 | 1.10 | 54.66 |
| 640.30 | -.50 | 0.00 | 521.2 | .192E+00 | 1.15 | 55.59 |
| 656.27 | -.50 | 0.00 | 551.6 | .181E+00 | 1.19 | 56.52 |
| 672.24 | -.50 | 0.00 | 583.0 | .172E+00 | 1.24 | 57.45 |
| 688.21 | -.50 | 0.00 | 615.5 | .162E+00 | 1.29 | 58.36 |
| 704.18 | -.50 | 0.00 | 649.0 | .154E+00 | 1.34 | 59.27 |
| 720.15 | -.50 | 0.00 | 683.5 | .146E+00 | 1.39 | 60.17 |
| 736.12 | -.50 | 0.00 | 719.0 | .139E+00 | 1.44 | 61.07 |
| 752.09 | -.50 | 0.00 | 755.7 | .132E+00 | 1.49 | 61.96 |
| 768.06 | -.50 | 0.00 | 793.3 | .126E+00 | 1.54 | 62.84 |
| 784.03 | -.50 | 0.00 | 832.1 | .120E+00 | 1.60 | 63.72 |
| 800.00 | -.50 | 0.00 | 871.9 | .115E+00 | 1.65 | 64.59 |

Cumulative travel time = 4465. sec

Simulation limit based on maximum specified distance = 800.00 m.

```
  This is the REGION OF INTEREST limitation.

END OF MOD341: BUOYANT AMBIENT SPREADING
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
CORMIX3: Buoyant Surface Discharges                     End of Prediction File
```

## *High Water*

```
CORMIX3 PREDICTION FILE:

                     CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX3:                                     Subsystem version:
 Buoyant Surface Discharges                  CORMIX_v.3.20_____September_1996
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
CASE DESCRIPTION
 Site name/label:          City^of^Toledo^Yaquina^River^HW
 Design case:              HW^0.5^MGD
 FILE NAME:                cormix\sim\HW105    .cx3
 Time of Fortran run:      08/29/05--14:42:25

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS    =      82.00  AS    =    566.62  QA    =      29.11  ICHREG= 1
 HA    =       6.91  HD    =      6.91
 UA    =       .051  F     =      .016  USTAR = .2332E-02
 UW    =      3.700  UWSTAR= .4235E-02
 Uniform density environment
 STRCND= U           RHOAM = 1015.0000

DISCHARGE PARAMETERS (metric units)
 BANK  = RIGHT       DISTB =       .50  Configuration: protruding_discharge
 SIGMA =      90.00  HD0   =       .46  SLOPE =      42.00
 Circular discharge pipe:
 D0    =       .457  A0    =      .102
 Dimensions of equivalent rectangular discharge:
 B0    =       .359  H0    =      .457  A0    = .1640E+00  AR    =      1.273
 Reduced channel geometry due to intrusion:
 B0    =       .359  H0    =      .284  A0    = .1019E+00  AR    =       .791
 (All relevant parameters further below are based on this geometry.)
 U0    =       .215  Q0    =      .022        = .2189E-01
 RHO0  =   998.5406  DRHO0 = .1646E+02  GP0   = .1590E+00
 C0    = .1000E+03   CUNITS= ppm
 IPOLL = 1           KS    = .0000E+00  KD    = .0000E+00

FLUX VARIABLES (metric units)
 Q0    = .2189E-01   M0    = .4702E-02  J0    = .3482E-02
 Associated length scales (meters)
 LQ    =        .32  LM    =       .30  Lm    =       1.33  Lb    =     25.68

NON-DIMENSIONAL PARAMETERS
 FR0   =        .95  FRCH  =      1.00  R     =       4.18

FLOW CLASSIFICATION
 333333333333333333333333333333333333333333
 3  Flow class (CORMIX3)       =     PL1    3
 3  Applicable layer depth HS =      6.91   3
 333333333333333333333333333333333333333333

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0    = .1000E+03   CUNITS= ppm
 NTOX  =  0
 NSTD  =  0
```

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| STATION NO. YAQUINA R C. MILL CR | 19-May | 1020 | 13 | | 0 | | | 27 | 2 | |
| H20 VALVE NO "5" | 18-May | 1320 | 16 | | 0 | | | 4.5 | <2 | |
| TRIB T-25 | 19-May | 1200 | 14 | | | | | ~7 | | |
| TRIB T-26 | 19-May | 1212 | 14.5 | | | | | 33 | | |
| TRIB T-27 | 19-May | 1221 | 14.5 | | | | | 2400 | | |
| TRIB T-28 | 19-May | 1230 | 12.5 | | | | | | | |
| TRIB T-29 | 19-May | 1240 | 12.5 | | | | | <2 | | |
| STATION NO ~'J | 19-May | 1006 | | | | | | 540 | | |
| STATION NO. 20 | 19-May | 1000 | | | | | | ~70 | | |
| YAQUINA R. COUNTY PARK | 19-May | 1028 | 13 | | 0 | | | 6.8 | 2 | |
| YAQUINA R. COUNTY PARK | 20-May | 1030 | 13 | | 0 | | | 23 | | |
| YAQUINA R. COUNTY PARK | 23-May | 1356 | 12 | | 0 | | | 140 | | |
| YAQUINA HATCHERY | 20-May | | 17.5 | | | | | 700 | 2 | |
| DRAIN DITCH ST 1 | 22-May | 1338 | | | | | | 54000 | | |

APPENDIX B

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| TOLEDO INFLUENT | | | | | | | | 6 | | |
| | | | | | | | | | | |
| STATION NO. TOLEDO EFFLUENT | 14-May | 1145 | | | | | | 22 | | 1.5 |
| STATION NO. TOLEDO EFFLUENT | 15-May | 1545 | | | | | | 330 | | 0.15 |
| STATION NO. TOLEDO EFFLUENT | 17-May | 1435 | | | | | | 2 | | 2.5 |
| STATION NO. TOLEDO EFFLUENT | 18-May | 1245E | | | | | | 130 | | 1.2 |
| STATION NO. TOLEDO EFFLUENT | 19-May | 958 | | | | | | <2 | | 2.3 |
| STATION NO. TOLEDO EFFLUENT | 20-May | 945 | | | | | | <2 | | 1.5 |
| STATION NO. TOLEDO EFFLUENT | 21-May | 1048 | | | | | | 2 | | 2.5 |
| STATION NO. TOLEDO EFFLUENT | 22-May | 1358 | | | | | | 330 | | 1.3 |
| STATION NO. TOLEDO EFFLUENT | 23-May | 1315 | | | | | | 1.8 | | 1.5 |
| | | | | | | | | | | |
| STATION NO. YAQUINA R C. | 18-May | 1245 | 14 | | 0 | | | 23 | | |

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| OLI | | | | | | | | | | |
| STATION NO. OLI | 19-May | 1053 | 14.5 | | 0 | | | 17 | <2 | |
| STATION NO. OLI | 20-May | 1049 | 13.5 | | 0 | | | 33 | 1.8 | |
| STATION NO. OLI | 21-May | 1250 | 13.5 | | 0 | | | 49 | | |
| STATION NO. OLI | 22-May | 1408 | 13.5 | | 0 | | | 23 | | |
| STATION NO. OLI | 23-May | 1340 | 13 | | 0 | | | 330 | | |
| | | | | | | | | | | |
| STATION NO. OL2 | 15-May | 1042 | 11 | | 0 | | | 79 | | |
| STATION NO. OL2 | 17-May | 1443 | 14 | | 0 | | | 33 | 3.6 | |
| STATION NO. OL2 | 18-May | 1225 | 13 | | 0 | | | 79 | 49 | |
| STATION NO. OL2 | 19-May | 1108 | 12.5 | | 0 | | | 170 | 13 | |
| STATION NO. OL2 | 20-May | 1104 | 12 | | 0 | | | 330 | 7.8 | |
| STATION NO. OL2 | 21-May | 1301 | 11.5 | | 0 | | | 130 | | |
| | | | | | | | | | | |
| STATION NO. TOLEDO INFLUENT | 14-May | 1140 | | | | | | 4.90E+0 5 | | |
| STATION NO. TOLEDO INFLUENT | 17-May | 1435 | | | | | | 2.30E+0 6 | | |
| STATION NO. | 22-May | 1352 | | | | | | 2.30E+0 | | |

APPENDIX B

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| STATION NO. D1 | 15-May | 945 | 12 | | 0 | | | 330 | | |
| STATION NO. D1 | 16-May | 1540 | 14 | | | | | 240 | 9.3 | |
| STATION NO. D1 | 17-May | 1459 | 14 | | 0.3 | | | 920 | 7.8 | |
| STATION NO. D1 | 18-May | 1105 | 11. 5 | | 0 | | | 110 | 2 | |
| STATION NO. D1 | 19-May | 1136 | 12.5 | | 0 | | | 180 | | |
| STATION NO. D1 | 20-May | 1118 | 11.5 | | 0 | | | 490 | 49 | |
| STATION NO. D1 | 21-May | 1234 | 12 | | 0 | | | 130 | | |
| STATION NO. D1 | 22-May | 1328 | 12 | | 0 | | | 330 | | |
| STATION NO. D1 | 23-May | 1415 | 11.5 | | 0 | | | 490 | | |
| | | | | | | | | | | |
| STATION NO. D2 | 15-May | 1003 | 11 | | 0 | | | 79 | | |
| STATION NO. D2 | 17-May | 1420 | 13.3 | | 0.2 | | | 130 | 11 | |
| STATION NO. D2 Little Beaver Creek | 18-May | 1120 | 11.5 | | 0 | | | 70 | <2 | |
| STATION NO. D2 Little Beaver Creek | 19-May | 1147 | 12 | | 0 | | | 170 | 2 | |
| STATION NO. D2 Little Beaver Creek | 20-May | 1130 | 11.5 | | 0 | | | 130 | 4.5 | |
| STATION NO. D2 Little Beaver Creek | 21-May | 1315 | 13 | | 0 | | | 49 | | |
| | | | | | | | | | | |
| STATION NO. OLI | 15-May | 1027 | 12.5 | | 0 | | | 79 | | |
| STATION NO. OLI | 16-May | 1525 | 16 | | 0 | | | 17 | <2 | |
| STATION NO. OLI | 17-May | 1435 | 17.5 | | 0 | | | 6.8 | <2 | |
| STATION NO. | 18-May | 1210 | 15 | | 0 | | | 33 | <2 | |

# APPENDIX B

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | |
| STATION NO. 18 | 14-May | 1530 | 12.4 | 12.4 | 0.4 | 0.4 | | 240 | | |
| STATION NO. 18 | 16-May | 1300 | 12.9 | 12.9 | 0.4 | 0.4 | | 110 | | |
| STATION NO. 18 | 19-May | 940 | 13.2 | | 0.1 | | | 33 | | |
| STATION NO. 18 | 20-May | 1204 | 12.5 | | 0 | | | 22 | | |
| STATION NO. 18 | 21-May | 1135 | 13.3 | | 0 | | | 17 | | |
| STATION NO. 18 | 22-May | 1240 | 12.7 | | 0 | | | 17 | | |
| STATION NO. 18 | 23-May | 1245 | 13.2 | | 1 | | | 33 | | |
| | | | | | | | | | | |
| STATION NO. B | 15-May | 1225 | 15.5 | | 0 | | | 14 | | |
| STATION NO. B Boone Slough | 16-May | 1555 | 17 | | | | | 4 | <2 | |
| STATION NO. B | 18-May | 1030 | 17 | | 0.75 | | | 2 | <2 | |
| STATION NO. B Boone Slough | 19-May | 924 | 17 | | 0 | | | 7.8 | 1.8 | |
| STATION NO. B Boone Slough | 20-May | 1208 | 17.5 | | 0 | | | 33 | 2 | |
| STATION NO. B Boone Slough | 21-May | 1212 | 18.5 | | 0 | | | 21 | | |
| | | | | | | | | | | |
| STATION NO. N | 15-May | 1210 | 14.5 | | 0.5 | | | 490 | | |
| STATION NO. N | 17-May | 1515 | 16.5 | | 7.0 | | | 33 | 2 | |
| STATION NO. N Nute Slough | 18-May | 1015 | 17 | | 0 | | | 170 | 49 | |
| STATION NO. N Nute Slough | 19-May | 937 | 17 | | 0 | | | 110 | 33 | |
| STATION NO. N Nute Slough | 20-May | 1150 | 16 | | 0 | | | 490 | 33 | |
| STATION NO. N Nute Slough | 21-May | 1225 | 16 | | 0 | | | 540 | 46 | |
| STATION NO. N Nute Slough | 23-May | 1429 | 15 | | 0 | | | 330 | | |
| | | | | | | | | | | |

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| STATION NO. 10 | 16-May | 1325 | | 14 | | 8.7 | | 31 | | |
| STATION NO. 10 | 17-May | 955 | 12.4 | 12.4 | 0.4 | 0.4 | | 49 | | |
| STATION NO. 10 | 18-May | 942 | 13.2 | 13.2 | 0.6 | 0.6 | | 240 | | |
| STATION NO. 10 | 18-May | 1329 | 15.4 | 14.6 | 2.6 | 4.7 | | 79 | | |
| STATION NO. 10 | 19-May | 915 | 14.6 | | 4.7 | | | 79 | | |
| STATION NO. 10 | 20-May | 1145 | 14 | | 2.1 | | | 23 | | |
| STATION NO. 10 | 21-May | 1155 | 14.7 | | 5.5 | | | 49 | | |
| STATION NO. 10 | 22-May | 1255 | 14.2 | | 6.6 | | | 33 | | |
| STATION NO. 10 | 23-May | 1302 | 14.3 | | 6.1 | | | 17 | | |
| | | | | | | | | | | |
| STATION NO. | 14-May | 1150 | 12.8 | 12.8 | 5 | 5.5 | | 33 | | |
| STATION NO. | 14-May | 1548 | 13.2 | 13.2 | 3.3 | 4.8 | | 70 | | |
| STATION NO. | 15-May | 1328 | 14.3 | 13.8 | 6.5 | 7.9 | | 49 | | |
| STATION NO. | 16-May | 1316 | 13.9 | 13.9 | 2.3 | 3.8 | | 33 | | |
| STATION NO. | 17-May | 1000 | 12.1 | 12.2 | 0.2 | 0.3 | | 79 | | |
| STATION NO. | 18-May | 1002 | | 13 | | 0.4 | | 130 | | |
| STATION NO. | 18-May | 1002 | 13.2 | | 0.4 | | | 33 | | |
| STATION NO. | 19-May | 920E | 14.2 | | 2.1 | | | 70 | | |
| STATION NO. | 20-May | 1149 | 13.5 | | 1.2 | | | 170 | | |
| STATION NO. | 21-May | 1147 | 14.3 | | 3.1 | | | 49 | | |
| STATION NO. | 22-May | 1250 | 13.9 | | 3.5 | | | 33 | | |
| STATION NO. | 23-May | 1259 | 14.4 | | 4.8 | | | 170 | | |
| | | | | | | | | | | |
| STATION NO. Q17 | 14-May | 1201 | 12.5 | 12.3 | 1.1 | 1.4 | | 140 | | |
| STATION NO. | 14-May | 1536 | 12.6 | 12.6 | 0.7 | .7 | | 49 | | |
| STATION NO. | 16-May | 1307 | 13.2 | 13.2 | 0.7 | 0.9 | | 70 | | |
| STATION NO. | 18-May | 1014 | 12.5 | 12.5 | 0.2 | 0.2 | | 23 | | |
| STATION NO. | 19-May | 925 | | 13.4 | | 0.4 | | 33 | | |
| STATION NO. | 19-May | 925 | 13.5 | | 0.5 | | | 49 | | |
| STATION NO. | 20-May | 1159 | 12.9 | | 0.2 | | | 170 | | |
| STATION NO. | 21-May | 1139 | 13.7 | | 0 | | | 49 | | |
| STATION NO. | 22-May | 1245 | 13.2 | | 1.8 | | | 33 | | |
| STATION NO. | 23-May | 1251 | 13.3 | | 2.3 | | | 130 | | |

APPENDIX B

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| STATION NO. Q8 | 16-May | 1338 | | 13.7 | | 20 | | 23 | <2 | |
| STATION NO. Q8 | 17-May | 935 | 13.2 | 13.2 | 1.6 | 1.6 | | 110 | 11 | |
| STATION NO. Q8 | 17-May | 1330 | 15.2 | 14.8 | 13.5 | 14.6 | | 4.5 | <2 | |
| STATION NO. Q8 | 18-May | 933 | 14.2 | 14.2 | 4 | 4 | | 110 | 11 | |
| STATION NO. Q8 | 18-May | 1338 | 14.4 | 14.6 | 13.8 | 14.7 | | 49 | <2 | |
| STATION NO. Q8 | 19-May | 903 | 14.6 | | 10 | | | 21 | <2 | |
| STATION NO. Q8 | 19-May | 1320 | 14.6 | | 12.5 | | | 13 | <2 | |
| STATION NO. Q8 | 20-May | 1133 | 14.3 | | 8.5 | | | 64 | 1.8 | |
| STATION NO. Q8 | 21-May | 1205 | 14.8 | | 9.8 | | | 17 | 4.5 | |
| STATION NO. Q8 | 22-May | 931 | 14.3 | | 9.3 | | | 17 | | |
| STATION NO. Q8 | 22-May | 1305 | 14.3 | | 9.5 | | | 13 | | |
| STATION NO. Q8 | 23-May | 940 | 13.5 | | 10.9 | | | 49 | | |
| STATION NO. Q8 | 23-May | 1310 | 14.5 | | 9 | | | 49 | | |
| | | | | | | | | | | |
| STATION NO. Q9 | 14-May | 1140 | 13.2 | 13.1 | 12.3 | 13.6 | | 26 | | |
| STATION NO. Q9 | 14-May | 1558 | 13.7 | 13.6 | 7.4 | 10.9 | | 33 | | |
| STATION NO. Q9 | 15-May | 1025 | 12.9 | 12.8 | 2.4 | 3 | | 220 | | |
| STATION NO. Q9 | 15-May | 1342 | 13.9 | 13.5 | 13.6 | 16.8 | | 49 | | |
| STATION NO. Q9 | 16-May | 1333 | 13.9 | 13.9 | 12.8 | 14.2 | | 49 | | |
| STATION NO. Q9 | 17-May | 948 | 13.1 | 12.9 | 0.8 | 0.9 | | 130 | | |
| STATION NO. Q9 | 18-May | 940 | | 14.1 | | 5.3 | | 240 | | |
| STATION NO. Q9 | 18-May | 940 | 13.8 | | 1.5 | | | 79 | | |
| STATION NO. Q9 | 19-May | 910 | 14.8 | | 8.1 | | | 49 | | |
| STATION NO. Q9 | 20-May | 1139 | 14.3 | | 6 | | | 49 | | |
| STATION NO. Q9 | 21-May | 1200 | 14.8 | | 8.3 | | | 70 | | |
| STATION NO. Q9 | 22-May | 1300 | 14.3 | | 8.1 | | | 49 | | |
| STATION NO. Q9 | 23-May | 1306 | 14.5 | | 8.6 | | | 79 | | |
| | | | | | | | | | | |
| STATION NO. 10 | 14-May | 1145 | 13 | 13 | 8.3 | 8.9 | | 49 | | |
| STATION NO. 10 | 14-May | 1553 | 13.2 | 13.3 | 4.3 | 8.9 | | 70 | | |
| STATION NO. 10 | 15-May | 1335 | 14.2 | 13.8 | 10.1 | 11. 1 | | 23 | | |
| STATION NO. 10 | 16-May | 1325 | 14.1 | | 8.1 | | | 33 | | |

APPENDIX B

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| STATION NO. Q5 | 21-May | 1220E | 14.9 | | 12.7 | | | 11 | <2 | |
| STATION NO. Q5 | 22-May | 922 | 14 | | 11.4 | | | 4.5 | | |
| STATION NO. Q5 | 22-May | 1310 | 14.3 | | 11.9 | | | 33 | | |
| STATION NO. Q5 | 23-May | 931 | 13.5 | | 14.2 | | | 13 | | |
| STATION NO. Q5 | 23-May | 1317 | 14.7 | | 10 | | | 23 | | |
| | | | | | | | | | | |
| STATION NO. Q6 | 14-May | 11:28 | 13.2 | 13.2 | 14.6 | 16.5 | | 23 | | |
| STATION NO. Q6 | 14-May | 1611 | 13.8 | 13.1 | 14.4 | 23.3 | | 13 | | |
| STATION NO. Q6 | 15-May | 1012 | 13.1 | | 9.1 | | | 31 | | |
| STATION NO. Q6 | 15-May | 1009 | | 13 | | 10.1 | | 49 | | |
| STATION NO. Q6 | 15-May | 1355 | 13.5 | 13.1 | 23.9 | 25.6 | | 4.5 | | |
| STATION NO. Q6 | 16-May | 929 | 13.1 | 13.1 | 2.2 | 4.7 | | 79 | 4.5 | |
| STATION NO. Q6 | 16-May | 1350 | 13.9 | 13.5 | 22.2 | 23.7 | | 2 | <2 | |
| STATION NO. Q6 | 17-May | 925 | 13.6 | 13.7 | 3.3 | 4.5 | | 79 | 7.8 | |
| STATION NO. Q6 | 17-May | 1345 | 14.7 | 14.2 | 20.5 | 21.1 | | 11 | <2 | |
| STATION NO. Q6 | 18-May | 925 | 14.4 | 14.4 | 7.5 | 7.8 | | 130 | | |
| STATION NO. Q6 | 18-May | 1343 | 15.1 | 14.4 | 17 | 18.4 | | 7.8 | | |
| STATION NO. Q6 | 19-May | 858 | 14.6 | | 12.7 | | | 14 | <2 | |
| STATION NO. Q6 | 19-May | 1330E | 14.4 | | 16 | | | 4.5 | <2 | |
| STATION NO. Q6 | 20-May | 1128 | 14.6 | | 10.6 | | | 17 | 4 | |
| STATION NO. Q6 | 21-May | 1212 | 14.9 | | 12 | | | 13 | 2 | |
| STATION NO. Q6 | 22-May | 925 | 14 | | 10.9 | | | 17 | | |
| STATION NO. Q6 | 22-May | 1308 | 14.2 | | 11.3 | | | 23 | | |
| STATION NO. Q6 | 23-May | 934 | 14 | | 15.4 | | | <2 | | |
| STATION NO. Q6 | 23-May | 1314 | 14.7 | | 9.6 | | | 23 | | |
| | | | | | | | | | | |
| STATION NO. Q8 | 14-May | 1136 | 13.2 | 13.2 | 14.6 | 17.3 | | 33 | | |
| STATION NO. Q8 | 14-May | 1604 | 13.7 | 13.2 | 10.3 | 19.4 | | 79 | | |
| STATION NO. Q8 | 15-May | 1016 | 12.9 | 12.9 | 4 | 5.2 | | 130 | | |
| STATION NO. Q8 | 15-May | 1349 | 13.9 | 13.5 | 19.3 | 21.2 | | 7.8 | | |
| STATION NO. Q8 | 16-May | 938 | 13 | 12.7 | 1.2 | 2 | | 49 | 4.5 | |
| STATION NO. Q8 | 16-May | 1340 | 14 | | 16.9 | | | 6.8 | <2 | |

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| STATION NO. 3W | 17-May | 1405 | 14.9 | 12.2 | 27.4 | 31.7 | | <2 | | |
| STATION NO. 3W | 18-May | 914 | 14.2 | 14 | 16.2 | 18.3 | | 23 | | |
| STATION NO. 3W | 18-May | 1403 | 13.4 | 13.1 | 27.2 | 28 | | 4 | | |
| STATION NO. 3W | 19-May | 845 | 14.1 | | 19.2 | | | 17 | | |
| STATION NO. 3W | 19-May | 1338 | 14.2 | | 22.4 | | | 4.5 | | |
| STATION NO. 3W | 20-May | 1116 | 14.2 | | 15.9 | | | 17 | | |
| STATION NO. 3W | 21-May | 1244 | 15 | | 15.8 | | | 4.5 | | |
| STATION NO. 3W | 22-May | 912 | 13.7 | | 20.6 | | | 2 | | |
| STATION NO. 3W | 22-May | 1320 | 14.2 | | 15.7 | | | 17 | | |
| STATION NO. 3W | 23-May | 924 | 13.3 | | 23.7 | | | 2 | | |
| STATION NO. 3W | 24-May | 1327 | 15 | | 16.3 | | | 7.8 | | |
| | | | | | | | | | | |
| STATION NO. Q5 | 14-May | 11:22 | 12.9 | 12.8 | 20.5 | 23 | | 12 | | |
| STATION NO. Q5 | 14-May | 1615 | 13.8 | 13.2 | 17.1 | 23.2 | | 23 | | |
| STATION NO. Q5 | 15-May | 1003 | 13.4 | 13 | 7.4 | 11.9 | | 13 | | |
| STATION NO. Q5 | 15-May | | BOTTOM | | | | | 170 | | |
| STATION NO. Q5 | 16-May | 925 | 13.1 | 13.1 | 3.6 | 7.4 | | 49 | 1.8 | |
| STATION NO. Q5 | 17-May | 921 | 13.8 | 13.8 | 4.'8 | 5.6 | | 33 | | |
| STATION NO. Q5 | 18-May | 1347 | 14.8 | 13.8 | 19.2 | 23 | | 2 | <2 | |
| STATION NO. Q5 | 19-May | 855 | 14.6 | | 12.7 | | | 13 | 4.5 | |
| STATION NO. Q5 | 20-May | 1126 | 14.3 | | 11.9 | | | 27 | 4.5 | |

APPENDIX B

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity, %, surface | Salinity, %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| STATION NO. 3E | 16-May | 901 | 13.3 | 13.2 | 11.5 | 16.9 | | 22 | | |
| STATION NO. 3E | 16-May | 1402 | 12.8 | 12 | 26.3 | 31.7 | | 4.5 | | |
| STATION NO. 3E | 17-May | 902 | | 13.8 | | 14.9 | | 11 | | |
| STATION NO. 3E | 17-May | 902 | 13.8 | | 13.8 | | | 49 | | |
| STATION NO. 3E | 17-May | 1400 | 13.6 | 12.2 | 26.5 | 31.5 | | <2 | | |
| STATION NO. 3E | 18-May | 911 | 14.1 | 14 | 16.6 | 16.6 | | 7.8 | | |
| STATION NO. 3E | 18-May | 1359 | 13.4 | 13.2 | 25.8 | 27.8 | | 2 | | |
| STATION NO. 3E | 19-May | 840 | 14.1 | | 19.2 | | | 13 | | |
| STATION NO. 3E | 19-May | 1335 | 13.9 | | 21.1 | | | 7.8 | | |
| STATION NO. 3E | 20-May | 1114 | 14.1 | | 16.3 | | | 49 | 33 | |
| STATION NO. 3E | 21-May | 1241 | 15 | | 15.5 | | | 3 1 | | |
| STATION NO. 3E | 22-May | 909 | 13.7 | | 17 | | | 4.5 | | |
| STATION NO. 3E | 22-May | 1318 | 14.2 | | 15.1 | | | 17 | | |
| STATION NO. 3E | 23-May | 921 | 13.4 | | 20.3 | | | 7.8 | | |
| STATION NO. 3E | 23-May | 1325 | 15 | | 14.7 | | | 13 | | |
| | | | | | | | | | | |
| STATION NO. 3W | 14-May | 11:11 | 12.8 | 12.4 | 26.2 | 28.8 | | 7.8 | | |
| STATION NO. 3W | 14-May | 1629 | 13.2 | 13.2 | 27.9 | 27.9 | | 17 | | |
| STATION NO. 3W | 15-May | 950 | | 12 | | 32 | | 17 | | |
| STATION NO. 3W | 15-May | 1409 | 12.9 | | 30.3 | | | 2 | | |
| STATION NO. 3W | 16-May | 910 | 13.3 | | 11. 7 | | | 23 | | |
| STATION NO. 3W | 16-May | 1406 | 13.3 | 12.1 | 30.8 | 31. 7 | | <2 | | |
| STATION NO. 3W | 17-May | 908 | 14 | | 13.7 | | | 22 | | |
| STATION NO. 3W | 17-May | 909 | | 14.9 | | 13.7 | | 7.8 | | |

**Table 21: Yaquina Bay field data May 1984 from Furfari (1985).**

| Station | Date 1984 | TIME | Temp, C surface | Temp, C bottom | Salinity , %, surface | Salinity , %, bottom | TC, MPN/ 100 ml | FC, MPN/ 100 ml | FS, MPN/ 100 ml | $Cl_2$ |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | |
| STATION NO. 1 | 14-May | 1040 | 12.1 | 12.1 | 31. 8 | | | <2 | | |
| STATION NO. 1 | 14-May | 1644 | 13.2 | 12.5 | 28.5 | 31 | | 1.8 | | |
| STATION NO. 1 | 15-May | 918 | 13 | 11. 8 | 24 | 31.5 | | 6.8 | | |
| STATION NO. 1 | 16-May | 847 | 13.2 | 12.6 | 21.1 | 26.1 | | 4.5 | | |
| STATION NO. 1 | 16-May | 1410E | 11.9 | 11. 5 | 32.4 | 32.4 | | <2 | | |
| STATION NO. 1 | 17-May | 850 | 13.4 | 12.9 | 21.9 | 25.8 | | 6.8 | | |
| STATION NO. 1 | 18-May | 852 | 13.4 | 12.9 | 24.1 | 28.5 | | <2 | | |
| STATION NO. 1 | 19-May | 830 | 13.4 | | 25.7 | | | 2 | | |
| STATION NO. 1 | 20-May | 1100 | 14 | | 21.5 | | | 7.8 | | |
| STATION NO. 1 | 21-May | 1256 | 14.2 | | 21.1 | | | 2 | | |
| STATION NO. 1 | 22-May | 855 | 12.9 | | 24.6 | | | 1.8 | | |
| STATION NO. 1 | 22-May | 1330 | 13.6 | | 20.2 | | | <2 | | |
| STATION NO. 1 | 23-May | 910 | 12.5 | | 26.7 | | | 22 | | |
| | | | | | | | | | | |
| STATION NO. 2 | 14-May | 1050E | 12.8 | 12.2 | 26.3 | 31.2 | | 22 | | |
| STATION NO. 2 | 14-May | 1634 | 13.2 | 12.6 | 26.3 | 29.8 | | 6.8 | | |
| STATION NO. 2 | 15-May | 931 | 13.2 | 13 | 15.3 | 22.3 | | 4.5 | | |
| STATION NO. 2 | 15-May | Botto | | | | | | 4.5 | | |
| STATION NO. 2 | 16-May | 859 | 13.3 | 13.2 | 14.5 | 17.8 | | 13 | | |
| STATION NO. 2 | 17-May | 1408 . | 12.5 | 11.7 | 31.5 | 32.4 | | 2 | | |
| STATION NO. 2 | 18-May | 906 | 14 | 13.4 | 18.1 | 24.4 | | 14 | | |
| STATION NO. 2 | 19-May | 1340 | 13.7 | | 25.8 | | | 13 | | |
| STATION NO. 2 | 20-May | 1110 | 14.3 | | 16.9 | | | 11 | | |
| STATION NO. 2 | 21-May | 1248 | 14.8 | | 16.2 | | | 23 | | |
| STATION NO. 2 | 22-May | 905 | 13.8 | | 21.5 | | | 2 | | |
| STATION NO. 2 | 22-May | 1323 | 14 | | 16.3 | | | 4.5 | | |
| STATION NO. 2 | 23-May | 918 | 13 | | 23.9 | | | 6.8 | | |
| | | | | | | | | | | |
| STATION NO. 3E | 14-May | 11:04 | 12.8 | 12.2 | 26.4 | 30.2 | | 7.8 | | |
| STATION NO. 3E | 14-May | 1626 | 13.3 | 13.1 | 24.7 | 25.6 | | 9.2 | | |
| STATION NO. 3E | 15-May | 945 | 13.3 | 13 | 16 | 23.9 | | 7.8 | | |
| STATION NO. 3E | 15-May | 1406 | 13.5 | 12 | 27.2 | 31. 7 | | 2 | | |

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| NEWPORT PACIFIC | 6-Dec | 1000 | | | | 110 | | |
| SPECIAL WATER SAMPLES AT OREGON OYSTER CO. | | | | | | | | |
| BAY WATER AT FLOAT | 30-Nov | 1055 | 9 | 3.5 | | 540 | | |
| BAY WATER AT FLOAT | 2-Dec | 941 | 8.5 | | | 79 | | |
| BAY WATER AT FLOAT | 5-Dec | 1420 | 9 | 15 | | 2 | | |
| SEEPAGE WEST OF PLANT | 30-Nov | 1052 | 10 | | | 920 | | |
| SEEPAGE WEST OF PLANT | 2-Dec | 1552 | 8.5 | 2 | | 460 | | |
| SEEPAGE WEST OF PLANT | 4-Dec | 1240 | | | | 22 | | |
| SEEPAGE WEST OF PLANT | 5-Dec | 1420 | | | | 130 | | |
| SEEPAGE EAST OF PLANT | 30-Nov | 1055 | 10.5 | 0 | | 4.5 | | |

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| C.Basin drain | 3-Dec | 1500 | | | | 2 | | |
| Drain Pipe - dredge spoil area | 29-Nov | 1215 | 8 | | | <1.8 | | |
| Tide Gate - dredge spoil area | 30-Nov | 1150 | 10.5 | 0.5 | | 920 | | |
| Tide Gate - dredge spoil area | 2-Dec | 1030 | | | 1300 | 33 | 49 | |
| (SAMPLES FROM BASKET HOLDING) | | | | | | | | |
| OREGON OYSTER CO. | 28-Nov | 1510 | | 1300 | 130 | | 45 | |
| OREGON OYSTER CO. | 29-Nov | 1018 | | - | 330 | 230 | | |
| OREGON OYSTER CO. | 30-Nov | - | | - | 230 | | 33 | |
| OREGON OYSTER CO. | 2-Dec | - | | - | 130 | | 23 | |
| OREGON OYSTER CO. | 3-Dec | 1052 | | 2400 | 110 | | 45 | |
| OREGON OYSTER CO. | 4-Dec | 1520 | | - | 45 | | - | |
| OREGON OYSTER CO. | 5-Dec | 1420 | | - | 78 | | - | |
| OREGON OYSTER CO. | 6-Dec | 1110 | | - | 330 | | - | |
| | | | | | | | | |
| FOWLER OYSTER CO. (SAMPLES FROM WET STORAGE TANK) | 29-Nov | | 9.5 | | | 45 | 700 | |
| FOWLER OYSTER CO. (SAMPLES FROM WET STORAGE TANK) | 3-Dec | 1035 | 10 | 230/00 | 340 | 45 | <18 | |
| FOWLER OYSTER CO. (SAMPLES FROM WET STORAGE TANK) | 4-Dec | - | 9 | 160/00 | | 20 | | |
| FOWLER OYSTER CO. (SAMPLES FROM WET STORAGE TANK) | 6-Dec | 1045 | | | - | <18 | - | |
| | | | | | | | | |
| NEWPORT PACIFIC | 6-Dec | 1000 | | | | 110 | | |
| NEWPORT PACIFIC | 6-Dec | 1000 | | | | 45 | | |

APPENDIX B

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| T-6 | 30-Nov | - | 9.5 | 3 | - | 49 | - | |
| T-8 | 30-Nov | 1035 | 9.5 | 2 | - | 350 | - | |
| T-9 | 30-Nov | - | 10.5 | 1 | - | 14 | - | |
| T-10 | 29-Nov | 1104 | 9 | 1.8 | 920 | 240 | 49 | |
| T-IO | 30-Nov | 1100 | 9 | 3 | - | 170 | - | |
| T-11 | 29-Nov | 1024 | 10.5 | - | - | 79 | - | |
| T-12 | 29-Nov | - | 10 | - | 920 | 350 | 14 | |
| T-12 | 1-Dec | 1040 | 9.5 | 1.6 | 540 | 350 | 23 | |
| T-12 | 6-Dec | 1100 | 10.5 | 4 | - | 17 | - | |
| T-13 | 1-Dec | 1050 | 10 | - | 240 | 49 | 13 | |
| T-18 | 29-Nov | . 1124 | 10 | - | - | 920 | - | |
| T-18 | 1-Dec | 1118 | 10.5 | - | - | 11300 | - | |
| T-26 | 28-Nov | 1330 | 10.8 | 0 | >1600 | 920 | 23 | |
| T-26 | 29-Nov | 1337 | 9.5 | 0.6 | - | 110 | - | |
| T-26 | 2-Dec | 1110 | 7.5 | 0 | 170 | 23 | 2 | |
| T-26 | 4-Dec | 1420 | 9.5 | 2 | - | 7.8 | - | |
| T-27 | 4-Dec | 1430 | 10 | - | - | 350 | - | |
| T-28 | 28-Nov | 1325 | 9.5 | 0 | 540 | 240 | 49 | |
| T-28 | 29-Nov | 1344 | 9.5 | 0.6 | - | 49 | - | |
| T-28 | 2-Dec | 1115 | 8 | 0 | 350 | 21 | 4.5 | |
| T-28 | 4-Dec | 1425 | 9 | 0 | - | 22 | - | |
| DRAIN PIPES IN TOLEDO | | | | | | | | |
| Pipe 1 - street drain 0 | 29-Nov | 1140 | 8.2 | 0 | | > 1600 | | |
| Pipe 1 street drain | 30-Nov | 1400 | 11.5 | | | 540 | | |
| | | | | | | | | |
| Pipe 1 street drain | 1-Dec | 1140 | 11 | | | 79 | | |
| Pipe 1 street drain | 2-Dec | 1040 | 8.5 | | 1100 | 170 | 13 | |
| Pipe 2 street drain | 30-Nov | 1400 | 13.5 | | | 64 | | |
| Pipe 3 street drain 5 | 30-Nov | 1400 | 11. 5 | | | 350 | | |

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| DEPOT CR 0-2 | 29-Nov | 1201 | 9 | - | - | 79 | - | |
| DEPOT CR 0-2 | 30-Nov | 1215 | 10.5 | 1.4 | 130 | 79 | 4 | |
| DEPOT CR 0-2 | 1-Dec | 1230 | 8.5 | - | 220 | 70 | 4.5 | |
| DEPOT CR 0-2 | 2-Dec | 1145 | 7.5 | - | - | 23 | - | |
| | | | | | | | | |
| LITTLE BEAVER CR | 28-Nov | 1408 | 9 | - | 920 | 79 | - | |
| | | | | | | | | |
| MOUTH DEPOT SL. | 3-Dec | 1411 | 6.6 | 0 | - | 33 | - | |
| MOUTH DEPOT SL. | 4-Dec | 1438 | 7 | 1.3 | - | 46 | - | |
| MOUTH DEPOT SL. | 5-Dec | 911 | 6.2 | 0.8 | - | 29 | - | |
| MOUTH DEPOT SL. | 6-Dec | 828 | 5.4 | 1 | - | 49 | - | |
| | | | | | | | | |
| BOONE SLOUGH - B | 28-Nov | 1434 | 8.5 | 0 | 1600 | 240 | - | |
| BOONE SLOUGH - B | 29-Nov | 1047 | 9 | 1 | 920 | 79 | 11 | |
| BOONE SLOUGH - B | 30-Nov | 1110 | 9 | 0.6 | - | 130 | - | |
| BOONE SLOUGH - B | 1-Dec | 1100 | 8.5 | 0 | 920 | 130 | 4.5 | |
| BOONE SLOUGH - B | 3-Dec | 1450 | 7.5 | - | 350 | 33 | 2 | |
| | | | | | | | | |
| NUTE SLOUGH - N | 28-Nov | 1438 | 9 | 0 | 1600 | 350 | - | |
| NUTE SLOUGH - N | 29-Nov | 1114 | 9 | 1 | 350 | 350 | 70 | |
| NUTE SLOUGH - N | 30-Nov | 1120 | 8.5 | 1 | >1600 | 170 | 33 | |
| NUTE SLOUGH - N | 1-Dec | 1112 | 8.5 | 0 | 920 | 110 | 22 | |
| NUTE SLOUGH - N | 3-Dec | 1445 | 8.5 | 0 | 920 | 33 | 2 | |
| MISCELLANEOUS TRIBUTARY AND MARSH DRAIN SAMPLES | | | | | | | | |
| T-3A | 29-Nov | 932 | - | - | - | 220 | - | |
| T-3A | 29-Nov | 1005 | - | - | - | 17 | - | |
| T-3A | 30-Nov | 1021 | 10.5 | - | - | 33 | - | |
| T-4 | 29-Nov | 1002 | 10.5 | - | 540 | 170 | 23 | |
| T-4 | 30-Nov | 1021 | 10 | - | - | 130 | - | |

APPENDIX B

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| TOLEDO STP - EFFLUENT | 28-Nov | 1130 | 11 | | <1.8 | <1. 8 | - | 1 |
| TOLEDO STP - EFFLUENT | 29-Nov | 1000 | 9.1 | | - | 6.8 | - | - |
| TOLEDO STP - EFFLUENT | 30-Nov | 1345 | 12.5 | | - | <1.8 | - | 0.5 |
| TOLEDO STP - EFFLUENT | 2-Dec | 1525 | - | | 13 | <1. 8 | - | 1.5 |
| TOLEDO STP - EFFLUENT | 3-Dec | 1346 | - | | 21 | 2 | <1.8 | - |
| TOLEDO STP - EFFLUENT | 4-Dec | 1435 | 12 | | - | 2 | - | 1.5 |
| TOLEDO STP - EFFLUENT | 5-Dec | 1530 | 11.5 | | - | 4.5 | - | 1.5 |
| | | | | | | | | |
| STAT 17A TOKYO SLOUGH | 4-Dec | 1440 | 7.3 | 1.8 | - | 49 | - | |
| GEORGIA PACIFIC EFFULENT | 5-Dec | 1044 | - | - | 7900 | 220 | 79 | |
| | | | | | | | | |
| OLALLA CR. OL-1 | 28-Nov | 1206 | 9 | - | - | 350 | | |
| OLALLA CR. OL-1 | 29-Nov | 1222 | 9 | 0.8 | 920 | 920 | 33 | |
| OLALLA CR. OL-1 | 30-Nov | 1200 | 10 | 1 | - | 79 | - | |
| OLALLA CR. OL-1 | 2-Dec | 1130 | 7.5 | 0 | 540 | 49 | - | |
| OLALLA CR. OL-1 | 5-Dec | 1520 | 7.5 | 0 | - | 49 | - | |
| | | | | | | | | |
| OLALLA CR. OL-2 | 28-Nov | 1235 | 9 | 0 | 920 | 49 | - | |
| OLALLA CR. OL-2 | 29-Nov | 1238 | 9 | - | - | 70 | - | |
| OLALLA CR. OL-2 | 30-Nov | 1210 | 10 | 0.5 | 280 | 33 | 23 | |
| OLALLA CR. OL-2 | 2-Dec | 1135 | 8.5 | - | - | 64 | - | |
| | | | | | | | | |
| DEPOT CR 0-1 | 28-Nov | 1355 | 8.5 | 0 | 280 | 130 | 33 | |
| DEPOT CR 0-1 | 29-Nov | 1144 | 9 | 1.2 | - | 79 | - | |
| DEPOT CR 0-1 | 30-Nov | 1140 | 10 | 1 | 920 | 79 | 23 | |
| DEPOT CR 0-1 | 1-Dec | 1130 | 9.5 | 0 | 130 | 33 | 11 | |
| DEPOT CR 0-1 | 2-Dec | 1015 | 7.5 | 0 | - | 33 | - | |
| DEPOT CR 0-1 | 5-Dec | 1450 | 7.5 | 0 | - | 79 | - | |
| DEPOT CR 0-1 | 6-Dec | 1450 | 7.5 | 0 | - | 33 | - | |
| | | | | | | | | |

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| YAQUINA R. @ MILLER CR | 1-Dec | 1347 | 8.4 | 0.5 | - | 27 | - | |
| YAQUINA R. @ BOAT RAMP - COUNTY PARK | 2-Dec | 1100 | 8 | 0 | 170 | 23 | - | |
| YAQUINA R. @ BOAT RAMP - COUNTY PARK | 3-Dec | 1430 | 8.5 | 0 | 130 | 17 | - | |
| YAQUINA R. @ BOAT RAMP - COUNTY PARK | 4-Dec | 1410 | 8.5 | 0 | - | 13 | - | |
| YAQUINA R. @ BOAT RAMP - COUNTY PARK | 5-Dec | 1510 | 8 | 0 | - | 49 | - | |
| YAQUINA R. @ BOAT RAMP - COUNTY PARK | 6-Dec | 1510 | 7.5 | 0 | - | 79 | - | |
| | | | | | | | | |
| STATION NO. 18 | 28-Nov | 1202 | 8.8 | 0.3 | 130 | 130 | - | |
| STATION NO. 18 | 30-Nov | 1221 | 9.1 | 0.5 | - | 79 | - | |
| STATION NO. 18 | 2-Dec | 1501 | 7.5 | 0.6 | 180 | 33 | 13 | |
| STATION NO. 18 | 3-Dec | 1030 | 6.5 | 0 | 280 | 46 | 13 | |
| STATION NO. 18 | 4-Dec | 1008 | 6.9 | 1.9 | - | 130 | - | |
| STATION NO. 18 | 5-Dec | 910 | 6.3 | 2.2 | - | 26 | - | |
| STATION NO. 18 | 6-Dec | 824 | 5.6 | 1.3 | - | 23 | - | |
| | | | | | | | | |
| TOLEDO STP - RAW | 28-Nov | 1134 | 12 | - | 240,000 | 130,000 | 7900 | |
| TOLEDO STP - RAW | 29-Nov | 1000 | - | - | - | >1600 | - | |
| TOLEDO STP - RAW | 30-Nov | 1345 | - | - | - | 1,600,000 | - | |
| TOLEDO STP - RAW | 1-Dec | 1150 | - | - | - | 920,000 | - | |
| TOLEDO STP - RAW | 2-Dec | 1525 | - | - | - | 540,000 | - | |
| | | | | | | | | |

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| STATION NO. 10 | 29-Nov | 1050 | 8.7 | 0.3 | - | 170 | - | |
| STATION NO. 10 | 30-Nov | 1206 | 8.7 | 0.5 | - | 110 | - | |
| STATION NO. 10 | 1-Dec | 1320 | 8.6 | 0.6 | 130 | 17 | 13 | |
| STATION NO. 10 | 2-Dec | 1447 | 7.5 | 0.7 | 220 | 79 | 13 | |
| STATION NO. 10 | 3-Dec | 1015 | 6.3 | 1.3 | >1600 - | 22 | 4.5 | |
| STATION NO. 10 | 1213 | 1422 | 6.9 | 1.3 | - | 49 | - | |
| STATION NO. 10 | 4-Dec | 952 | 7 | 9.1 | - | 23 | - | |
| STATION NO. 10 | 4-Dec | 1450 | 7.8 | 5.9 | - | 33 | - | |
| STATION NO. 10 | 5-Dec | 842 | 6.6 | 11 | - | 11 | - | |
| STATION NO. 10 | 6-Dec | 805 | 6.1 | 7.4 | - | 17 | - | |
| | | | | | | | | |
| STATION NO. Q15 | 28-Nov | 1148 | 8.8 | 0.4 | 540 | 350 | - | |
| STATION NO. Q15 | 29-Nov | 1055 | 8.6 | 0.3 | - | 79 | - | |
| STATION NO. Q15 | 30-Nov | 1210 | 8.8 | 0.5 | - | 110 | - | |
| STATION NO. Q15 | 1-Dec | 1334 | 8.6 | 0.6 | - | 17 | - | |
| STATION NO. Q15 | 2-Dec | 1450 | 7.5 | 0.6 | 130 | 23 | 7.8 | |
| STATION NO. Q15 | 3-Dec | 1025E | 6.4 | 0.3 | - | 33 | - | |
| STATION NO. Q15 | 4-Dec | 958 | 6.7 | 6.3 | - | 26 | - | |
| STATION NO. Q15 | 5-Dec | 845 | 6.5 | 9 | - | 22 | - | |
| STATION NO. Q15 | 6-Dec | 810 | 6 | 6.5 | - | 33 | - | |
| | | | | | | | | |
| STATION NO. Q17 | 28-Nov | 1200 | 8.8 | 0.2 | 540 | 350 | - | |
| STATION NO. Q17 | 30-Nov | 1216 | 9 | 0.5 | 170 | 79 | - | |
| STATION NO. Q17 | 1-Dec | 1340 | 8.6 | 0.5 | - | 33 | - | |
| STATION NO. Q17 | 2-Dec | 1457 | 7.5 | 0.6 | 110 | 17 | 7.8 | |
| STATION NO. Q17 | 3-Dec | 1030E | 6.5 | 0 | - | 33 | - | |
| STATION NO. Q17 | 4-Dec | 959 | 6.9 | 1.9 | - | 79 | - | |
| STATION NO. Q17 | 5-Dec | 858 | 6.3 | 3.7 | - | 170 | - | |
| STATION NO. Q17 | 6-Dec | 820 | 5.7 | 2.7 | - | 79 | - | |
| | | | | | | | | |

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| STATION NO. Q8 | 28-Nov | 1138 | 8.6 | 0.6 | 1600 | 350 | 240 | |
| STATION NO. Q8 | 29-Nov | 1035 | 8.6 | 0.4 | 920 | 110 | 70 | |
| STATION NO. Q8 | 30-Nov | 1158 | 8.7 | 0.9 | 350 | 240 | 33 | |
| STATION NO. Q8 | 1-Dec | 1323 | 8.6 | 0.7 | 280 | 32 | 13 | |
| STATION NO. Q8 | 2-Dec | 1439 | 7.8 | 3.2 | 33 | 23 | 4.5 | |
| STATION NO. Q8 | 3-Dec | 1005E | 6.7 | 4.4 | 170 | 49 | 23 | |
| STATION NO. Q8 | 3-Dec | 1433 | 7.1 | 3.4 | 350 | 130 | 2 | |
| STATION NO. Q8 | 4-Dec | 944 | 6.7 | 9.4 | - | 23 | 4.5 | |
| STATION NO. Q8 | 4-Dec | 1458 | 8 | 8.9 | - | 23 | - | |
| STATION NO. Q8 | 5-Dec | 826 | 6.5 | 13.6 | - | 23 | 2 | |
| STATION NO. Q8 | 5-Dec | 1543 | 7.4 | 9 | - | 33 | - | |
| STATION NO. Q8 | 6-Dec | 754 | 6.5 | 13 | - | 79 | - | |
| STATION NO. Q8 | 6-Dec | 1545 | 7.2 | 9.4 | - | 17 | - | |
| | | | | | | | | |
| STATION NO. Q9 | 28-Nov | 1141 | 8.8 | 0.5 | 350 | 350 | 49 | |
| STATION NO. Q9 | 29-Nov | 1105 | 8.6 | 0.4 | 920 | 280 | 23 | |
| STATION NO. Q9 | 30-Nov | 1201 | 8.7 | 0.5 | 140 | 110 | 23 | |
| STATION NO. Q9 | 1-Dec | 1328 | 8.6 | 0.7 | 170 | 79 | 4.5 | |
| STATION NO. Q9 | 2-Dec | 1442 | 7.7 | 2.2 | 79 | 49 | 23 | |
| STATION NO. Q9 | 3-Dec | 1040E | 6.7 | 4.2 | 220 | 17 | 13 | |
| STATION NO. Q9 | 3-Dec | 1429 | 7.1 | 3.4 | 540 | 13 | 2 | |
| STATION NO. Q9 | 4-Dec | 948 | 6.8 | 8.3 | 33 | 33 | <1.8 | |
| STATION NO. Q9 | 4-Dec | 1454 | 7.9 | 9 | - | 49 | - | |
| STATION NO. Q9 | 5-Dec | 832 | 7 | 12.4 | - | 11 | 7.8 | |
| STATION NO. Q9 | 5-Dec | 1548 | 7.4 | 8.6 | - | 17 | - | |
| STATION NO. Q9 | 6-Dec | 800 | 6.3 | 9.3 | - | 17 | - | |
| STATION NO. Q9 | 6-Dec | 1550 | 7 | 8.4 | - | 79 | 7.8 | |
| | | | | | | | | |
| STATION NO. 10 | 28-Nov | 1145 | 8.8 | 0.4 | 220 | 110 | - | |

APPENDIX B

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| | | | | | | | | |
| STATION NO. Q5 | 28-Nov | 1132 | 8.2 | 1.9 | 1600 | 220 | 79 | |
| STATION NO. Q5 | 29-Nov | 1021 | 8.6 | 0.9 | 1600 | 350 | 130 | |
| STATION NO. Q5 | 30-Nov | 1130 | 8.6 | 1.5 | 350 | 170 | 130 | |
| STATION NO. Q5 | 1-Dec | 1316 | 8.6 | 2.1 | 540 | 49 | 11 | |
| STATION NO. Q5 | 2-Dec | 1232 | 8 | 5.2 | 79 | 79 | 4.5 | |
| STATION NO. Q5 | 3-Dec | 950 | 6.7 | 5.7 | 170 | 49 | 23 | |
| STATION NO. Q5 | 3-Dec | 1439 | 7.3 | 5.1 | 130 | 49 | 4.5 | |
| STATION NO. Q5 | 4-Dec | 934 | 7.7 | 16.9 | | 17 | <1.8 | |
| STATION NO. Q5 | 4-Dec | 1503 | 8 | 10.8 | | 17 | | |
| STATION NO. Q5 | 5-Dec | 759 | 7 | 16.1 | 33 | 7.8 | 4.5 | |
| STATION NO. Q5 | 5-Dec | 1535 | | | | 31 | | |
| STATION NO. Q5 | 6-Dec | 739 | 6.6 | 14.6 | | 46 | | |
| STATION NO. Q5 | 6-Dec | 1536 | 7.5 | 13.4 | | 17 | | |
| | | | | | | | | |
| STATION NO. Q6 | 28-Nov | 1134 | 8.3 | 1.9 | 240 | 240 | 49 | |
| STATION NO. Q6 | 29-Nov | 1029 | 8.6 | 0.8 | 540 | 220 | 130 | |
| STATION NO. Q6 | 30-Nov | 1150 | 8.6 | 1.5 | 540 | 240 | 49 | |
| STATION NO. Q6 | 1-Dec | 1320 | 8.6 | 1.2 | 220 | 170 | 11 | |
| STATION NO. Q6 | 2-Dec | 1435 | 8 | 4.3 | 95 | 46 | 4.5 | |
| STATION NO. Q6 | 3-Dec | 958 | 6.5 | 4.8 | 220 | 23 | 4.5 | |
| STATION NO. Q6 | 3-Dec | 1436 | 7.3 | 5.4 | - | 22 | <1. 8 | |
| STATION NO. Q6 | 4-Dec | 941 | 7.2 | 13 | 33 | 13 | 11 | |
| STATION NO. Q6 | 4-Dec | 1500 | 8 | 10.8 | - | 23 | - | |
| STATION NO. Q6 | 5-Dec | 812 | 7.4 | 17.2 | - | 23 | 4.5 | |
| STATION NO. Q6 | 5-Dec | 1539 | 7.5 | 12.4 | - | 33 | - | |
| STATION NO. Q6 | 6-Dec | 748 | 6.5 | 13 | - | 27 | - | |
| STATION NO. Q6 | 6-Dec | 1540 | 7.5 | 13 | - | 33 | - | |
| | | | | | | | | |

APPENDIX B

Table 20: Yaquina Bay field data November and December 1984 from Furfari (1985).

| Sample location | DATE 1984 | TIME | TEMP | SAL | TC | FC | FS | Chlorine residual, mg/l |
|---|---|---|---|---|---|---|---|---|
| | | | °C | % | MPN/ 100 ml | MPN/ 100 ml | MPN/ 100 ml | |
| | | | | | | | | |
| STATION NO.1 | 28-Nov | 1115 | 8.8 | 16.4 | 240 | 130 | 17 | |
| STATION NO.1 | 29-Nov | 958 | 8.3 | 10.4 | | 140 | | |
| STATION NO.1 | 1-Dec | 1255 | 8.6 | 6.5 | | 140 | | |
| STATION NO.1 | 2-Dec | 1412 | 8.1 | 5.9 | | 70 | | |
| STATION NO.1 | 4-Dec | 1520 | 8.6 | 18.6 | | 13 | | |
| STATION NO.1 | 5-Dec | 733 | 10.1 | 32.8 | | <1.8 | | |
| STATION NO.1 | 6-Dec | 717 | 9.9 | 31.1 | | <1. 8 | | |
| | | | | | | | | |
| STATION NO. 2 | 28-Nov | 1120 | 8.3 | 7.2 | | 350 | | |
| STATION NO. 2 | 29-Nov | 1009 | 8.6 | 6.7 | 540 | 170 | | |
| STATION NO. 2 | 1-Dec | 1302 | 8.7 | 4 | | 49 | | |
| STATION NO. 2 | 2-Dec | 1422 | 8.2 | 9 | 350 | 79 | 4.5 | |
| STATION NO. 2 | 4-Dec | 1512 | 8.2 | 15.4 | | 6.8 | | |
| STATION NO. 2 | 5-Dec | 741 | 8 | 23.7 | | 17 | | |
| STATION NO. 2 | 6-Dec | 724 | 7.8 | 24.5 | | 17 | | |
| | | | | | | | | |
| STATION NO. 3 | 28-Nov | 1124 | 8.1 | 3.6 | | 540 | | |
| STATION NO. 3 | 29-Nov | 1010 | 8.5 | 4 | | 170 | | |
| STATION NO. 3 | 30-Nov | 1137 | 8.7 | 3.6 | | 49 | | |
| STATION NO. 3 | 1-Dec | 1305 | 8.7 | 3.7 | 79 | 33 | 13 | |
| STATION NO. 3 | 2-Dec | 1425 | 8.1 | 6.9 | | 79 | | |
| STATION NO. 3 | 3-Dec | 944 | 7.2 | 12.2 | | 79 | | |
| STATION NO. 3 | 3-Dec | 1448 | 7.4 | 7.7 | | 33 | | |
| STATION NO. 3 | 4-Dec | 926 | 7.6 | 22.2 | | 110 | | |
| STATION NO. 3 | 4-Dec | 1510 | 8.1 | 14.3 | | 11 | | |
| STATION NO. 3 | 5-Dec | 745 | 8.5 | 25.4 | | 7.8 | | |
| STATION NO. 3 | 6-Dec | 728 | 7.6 | 20.8 | | 7.8 | | |

## Appendix V – Field Data in Yaquina Bay 1984

This appendix includes summaries of some of the water quality field data from Furfari (1985). A map of the study areas is shown in Figure 84. Field stations used by Furfari (1985).Figure 84. Tables of the field data taken in 1984 are summarized in Table 20 and Table 21.

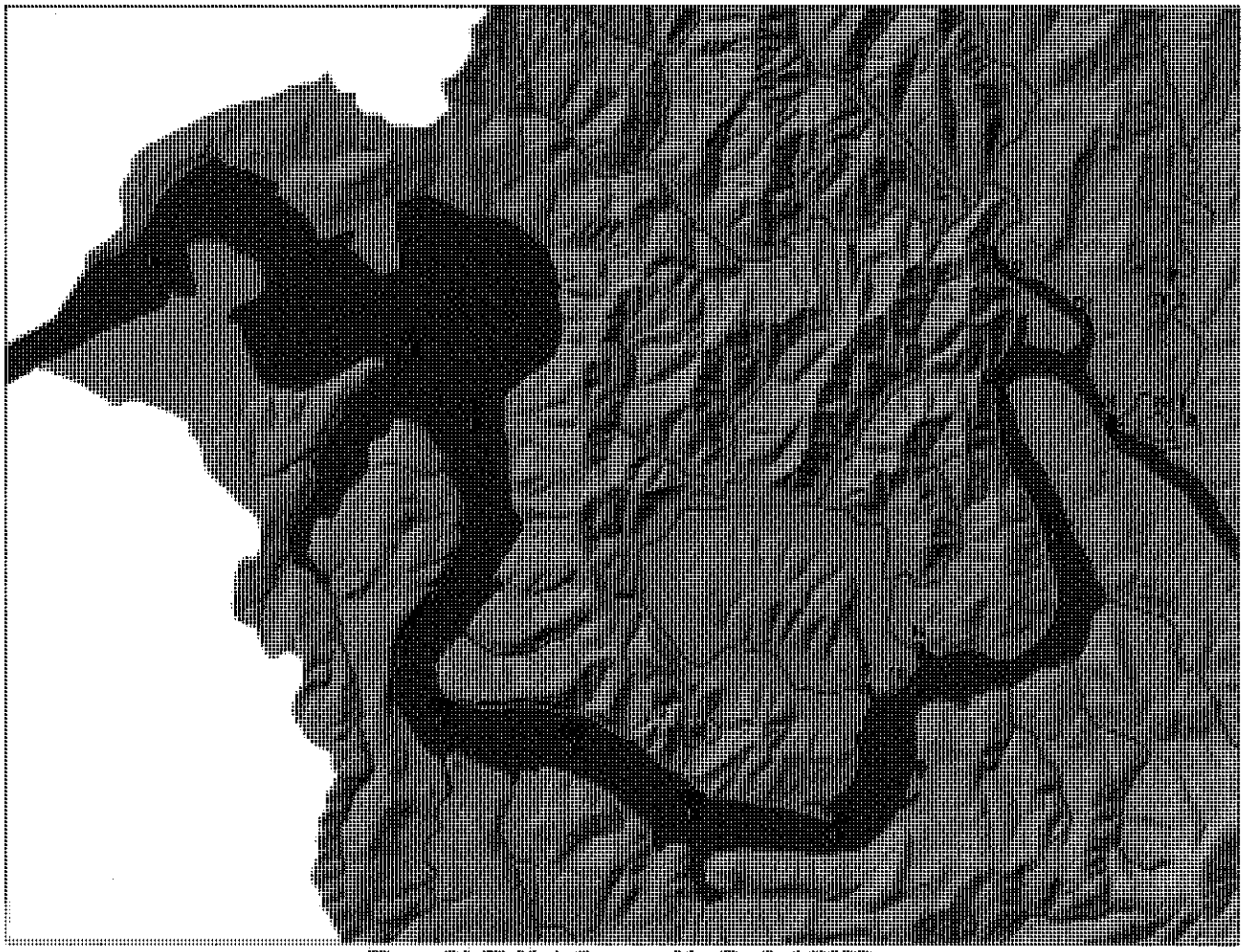

Figure 84. Field stations used by Furfari (1985).

| | | | | | | |
|---|---|---|---|---|---|---|
| 348.83 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 358.45 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 368.07 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 377.68 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 387.30 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 396.91 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 406.53 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 416.14 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 425.76 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 435.37 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 444.99 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 454.61 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 464.22 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 473.84 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 483.45 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 493.07 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 502.68 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 512.30 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 521.92 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 531.53 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 541.15 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 550.76 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 560.38 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 569.99 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 579.61 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 589.22 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 598.84 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 608.46 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 618.07 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 627.69 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 637.30 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 646.92 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 656.53 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 666.15 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 675.77 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 685.38 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 695.00 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 704.61 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 714.23 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 723.84 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 733.46 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 743.07 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 752.69 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 762.31 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 771.92 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 781.54 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 791.15 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 800.77 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 810.38 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |
| 820.00 | -.50 | 0.00 | 20.5 | .489E+01 | .07 | 82.00 |

Cumulative travel time = 10084. sec

Simulation limit based on maximum specified distance = 820.00 m.
This is the REGION OF INTEREST limitation.

END OF MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

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CORMIX3: Buoyant Surface Discharges — End of Prediction File

```
In this prediction interval the plume distance meets or exceeds
  the regulatory value =    62.00 m.
This is the extent of the REGULATORY MIXING ZONE.
    66.75      -.50     0.00      5.1  .194E+02     .04     31.44
    73.40      -.50     0.00      5.3  .188E+02     .04     33.21
    80.04      -.50     0.00      5.5  .182E+02     .04     34.92
    86.69      -.50     0.00      5.7  .177E+02     .04     36.56
    93.33      -.50     0.00      5.8  .171E+02     .04     38.16
    99.98      -.50     0.00      6.0  .166E+02     .04     39.70
   106.62      -.50     0.00      6.2  .160E+02     .04     41.21
   113.27      -.50     0.00      6.5  .155E+02     .04     42.67
   119.91      -.50     0.00      6.7  .150E+02     .04     44.10
   126.56      -.50     0.00      6.9  .145E+02     .04     45.50
   133.21      -.50     0.00      7.1  .140E+02     .04     46.87
   139.85      -.50     0.00      7.4  .135E+02     .04     48.21
   146.50      -.50     0.00      7.6  .131E+02     .04     49.53
   153.14      -.50     0.00      7.9  .126E+02     .04     50.82
   159.79      -.50     0.00      8.2  .122E+02     .04     52.10
   166.43      -.50     0.00      8.5  .118E+02     .04     53.35
   173.08      -.50     0.00      8.8  .114E+02     .04     54.59
   179.72      -.50     0.00      9.1  .110E+02     .04     55.81
   186.37      -.50     0.00      9.4  .106E+02     .04     57.02
   193.02      -.50     0.00      9.8  .102E+02     .05     58.21
   199.66      -.50     0.00     10.1  .989E+01     .05     59.39
   206.31      -.50     0.00     10.5  .955E+01     .05     60.55
   212.95      -.50     0.00     10.8  .922E+01     .05     61.70
   219.60      -.50     0.00     11.2  .890E+01     .05     62.85
   226.24      -.50     0.00     11.6  .860E+01     .05     63.98
   232.89      -.50     0.00     12.0  .830E+01     .05     65.10
   239.54      -.50     0.00     12.5  .802E+01     .05     66.21
   246.18      -.50     0.00     12.9  .775E+01     .05     67.31
   252.83      -.50     0.00     13.4  .749E+01     .05     68.40
   259.47      -.50     0.00     13.8  .724E+01     .05     69.49
   266.12      -.50     0.00     14.3  .700E+01     .05     70.57
   272.76      -.50     0.00     14.8  .676E+01     .06     71.64
   279.41      -.50     0.00     15.3  .654E+01     .06     72.70
   286.05      -.50     0.00     15.8  .633E+01     .06     73.76
   292.70      -.50     0.00     16.3  .612E+01     .06     74.81
   299.35      -.50     0.00     16.9  .592E+01     .06     75.85
   305.99      -.50     0.00     17.4  .573E+01     .06     76.89
   312.64      -.50     0.00     18.0  .555E+01     .06     77.93
   319.28      -.50     0.00     18.6  .537E+01     .06     78.95
   325.93      -.50     0.00     19.2  .520E+01     .06     79.98
   332.57      -.50     0.00     19.8  .504E+01     .07     80.99
   339.22      -.50     0.00     20.5  .489E+01     .07     82.00
Cumulative travel time =        4172. sec
Plume is LATERALLY FULLY MIXED at the end of the buoyant spreading regime.

END OF MOD341: BUOYANT AMBIENT SPREADING
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-----------------------------------------------------------------------------
BEGIN MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

  Vertical diffusivity (initial value)    =  .555E-02 m^2/s
  Horizontal diffusivity (initial value)  =  .694E-02 m^2/s

 Profile definitions:
   BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
      = or equal to water depth, if fully mixed
   BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
        measured horizontally in Y-direction
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

 Plume Stage 2 (bank attached):
       X          Y         Z         S        C         BV       BH
   339.22      -.50     0.00     20.5  .489E+01     .07     82.00
```

APPENDIX B

END OF MOD302: ZONE OF FLOW ESTABLISHMENT

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BEGIN MOD331: UPSTREAM INTRUDING PLUME

Control volume inflow:

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| .00 | .00 | 0.00 | 2.1 | .487E+02 | .46 | .79 |

UPSTREAM INTRUSION PROPERTIES:

| | | |
|---|---|---|
| Upstream intrusion length | = | 3.47 m |
| X-position of upstream stagnation point | = | -3.47 m |
| Thickness in intrusion region | = | .10 m |
| Half-width at downstream end | = | 9.35 m |
| Thickness at downstream end | = | .10 m |

Profile definitions:
BV = top-hat thickness,measured vertically
BH = top-hat half-width, measured horizontally from bank/shoreline
S = hydrodynamic average (bulk) dilution
C = average (bulk) concentration (includes reaction effects, if any)

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| -3.47 | .00 | 0.00 | 9999.9 | .000E+00 | .00 | .00 |
| -3.26 | .00 | 0.00 | 6.7 | .149E+02 | .03 | 1.32 |
| -2.24 | .00 | 0.00 | 2.8 | .354E+02 | .07 | 3.21 |
| -1.22 | .00 | 0.00 | 2.2 | .449E+02 | .09 | 4.35 |
| -.20 | .00 | 0.00 | 2.1 | .486E+02 | .10 | 5.24 |
| .82 | .00 | 0.00 | 2.1 | .470E+02 | .10 | 6.00 |
| 1.84 | .00 | 0.00 | 2.4 | .418E+02 | .10 | 6.68 |
| 2.86 | .00 | 0.00 | 2.7 | .366E+02 | .10 | 7.29 |
| 3.88 | .00 | 0.00 | 3.0 | .329E+02 | .10 | 7.86 |
| 4.90 | .00 | 0.00 | 3.2 | .308E+02 | .10 | 8.38 |
| 5.92 | .00 | 0.00 | 3.4 | .297E+02 | .10 | 8.88 |
| 6.94 | .00 | 0.00 | 3.4 | .291E+02 | .10 | 9.35 |

Cumulative travel time = 85. sec

END OF MOD331: UPSTREAM INTRUDING PLUME

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** End of NEAR-FIELD REGION (NFR) **

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BEGIN MOD341: BUOYANT AMBIENT SPREADING

Plume is ATTACHED to RIGHT bank/shore.
Plume width is now determined from RIGHT bank/shore.

Profile definitions:
BV = top-hat thickness,measured vertically
BH = top-hat half-width, measured horizontally from bank/shoreline
S = hydrodynamic average (bulk) dilution
C = average (bulk) concentration (includes reaction effects, if any)

Plume Stage 2 (bank attached):

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 6.94 | -.50 | 0.00 | 3.4 | .291E+02 | .10 | 9.35 |
| 13.59 | -.50 | 0.00 | 3.7 | .267E+02 | .08 | 12.90 |
| 20.23 | -.50 | 0.00 | 4.0 | .252E+02 | .07 | 15.94 |
| 26.88 | -.50 | 0.00 | 4.2 | .240E+02 | .06 | 18.67 |
| 33.52 | -.50 | 0.00 | 4.3 | .230E+02 | .06 | 21.15 |
| 40.17 | -.50 | 0.00 | 4.5 | .222E+02 | .05 | 23.46 |
| 46.81 | -.50 | 0.00 | 4.7 | .214E+02 | .05 | 25.62 |
| 53.46 | -.50 | 0.00 | 4.8 | .207E+02 | .05 | 27.66 |
| 60.10 | -.50 | 0.00 | 5.0 | .201E+02 | .05 | 29.60 |

** REGULATORY MIXING ZONE BOUNDARY **

```
Reduced channel geometry due to intrusion:
 B0     =      .359 H0      =       .280 A0     = .1006E+00  AR     =        .781
 (All relevant parameters further below are based on this geometry.)
 U0     =      .218 Q0      =       .022        = .2189E-01
 RHO0   =  998.3550 DRHO0 = .1748E+02  GP0    = .1687E+00
 C0     = .1000E+03 CUNITS=   ppm
 IPOLL  =  1        KS      = .0000E+00  KD     = .0000E+00

FLUX VARIABLES (metric units)
 Q0     = .2189E-01  M0     = .4767E-02  J0     = .3694E-02
 Associated length scales (meters)
 LQ     =       .32  LM     =       .30  Lm     =        .85  Lb     =       6.88

NON-DIMENSIONAL PARAMETERS
 FR0    =       .94  FRCH   =      1.00  R      =       2.67

FLOW CLASSIFICATION
 333333333333333333333333333333333333333333
 3  Flow class (CORMIX3)      =     PL1    3
 3  Applicable layer depth HS =     7.28   3
 333333333333333333333333333333333333333333

MIXING ZONE / TOXIC DILUTION / REGION OF INTEREST PARAMETERS
 C0     = .1000E+03  CUNITS=   ppm
 NTOX   =  0
 NSTD   =  0
 REGMZ  =  1
 REGSPC=   1          XREG   =     62.00  WREG   =        .00  AREG   =        .00
 XINT   =    820.00   XMAX   =    820.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the WATER SURFACE and at center of discharge
      channel/outlet:        .50 m  from the RIGHT bank/shore.
    X-axis points downstream
    Y-axis points to left as seen by an observer looking downstream
    Z-axis points vertically upward (in CORMIX3, all values Z = 0.00)
NSTEP  = 50 display intervals per module
-------------------------------------------------------------------------------
        TRJBUO     TRJATT     TRJBND     TRJNBY     TRJCOR     DILCOR
C        3.651      1.000       .996       .996      3.638      1.000
-------------------------------------------------------------------------------
BEGIN MOD301: DISCHARGE MODULE

 Efflux conditions:
       X          Y        Z          S         C         BV        BH
      .00        .00     0.00       1.0   .100E+03     .28       .18

END OF MOD301: DISCHARGE MODULE
-------------------------------------------------------------------------------
-------------------------------------------------------------------------------
BEGIN MOD302: ZONE OF FLOW ESTABLISHMENT

  Control volume inflow:
       X          Y        Z          S         C         BV        BH
      .00        .00     0.00       1.0   .100E+03     .28       .18

 VERTICAL MIXING occurs in the initial zone of flow establishment.
 Profile definitions:
   BV = Gaussian 1/e (37%) vertical thickness
   BH = Gaussian 1/e (37%) horizontal half-width, normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

  Control volume outflow:
       X          Y        Z          S         C         BV        BH
      .00        .00     0.00       2.1   .487E+02     .46       .79
 Cumulative travel time =            0. sec
```

```
 532.85      -.50    0.00    12.8   .782E+01    .07    82.00
 545.91      -.50    0.00    12.8   .782E+01    .07    82.00
 558.96      -.50    0.00    12.8   .782E+01    .07    82.00
 572.01      -.50    0.00    12.8   .782E+01    .07    82.00
 585.06      -.50    0.00    12.8   .782E+01    .07    82.00
 598.12      -.50    0.00    12.8   .782E+01    .07    82.00
 611.17      -.50    0.00    12.8   .782E+01    .07    82.00
 624.22      -.50    0.00    12.8   .782E+01    .07    82.00
 637.27      -.50    0.00    12.8   .782E+01    .07    82.00
 650.32      -.50    0.00    12.8   .782E+01    .07    82.00
 663.38      -.50    0.00    12.8   .782E+01    .07    82.00
 676.43      -.50    0.00    12.8   .782E+01    .07    82.00
 689.48      -.50    0.00    12.8   .782E+01    .07    82.00
 702.53      -.50    0.00    12.8   .782E+01    .07    82.00
 715.58      -.50    0.00    12.8   .782E+01    .07    82.00
 728.64      -.50    0.00    12.8   .782E+01    .07    82.00
 741.69      -.50    0.00    12.8   .782E+01    .07    82.00
 754.74      -.50    0.00    12.8   .782E+01    .07    82.00
 767.79      -.50    0.00    12.8   .782E+01    .07    82.00
 780.84      -.50    0.00    12.8   .782E+01    .07    82.00
 793.90      -.50    0.00    12.8   .782E+01    .07    82.00
 806.95      -.50    0.00    12.8   .782E+01    .07    82.00
 820.00      -.50    0.00    12.8   .782E+01    .07    82.00
Cumulative travel time =        15950. sec

Simulation limit based on maximum specified distance =     820.00 m.
  This is the REGION OF INTEREST limitation.

END OF MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT
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CORMIX3: Buoyant Surface Discharges                  End of Prediction File
```

## *High High Water*

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CORMIX3 PREDICTION FILE:

                      CORNELL MIXING ZONE EXPERT SYSTEM
Subsystem CORMIX3:                                         Subsystem version:
 Buoyant Surface Discharges                    CORMIX_v.3.20____September_1996
-----------------------------------------------------------------------------

-----------------------------------------------------------------------------

CASE DESCRIPTION
 Site name/label:          City^of^Toledo^Yaquina^River^HHW
 Design case:              HHW^0.5^MGD
 FILE NAME:                cormix\sim\HHW105   .cx3
 Time of Fortran run:      08/29/05--14:43:59

ENVIRONMENT PARAMETERS (metric units)
 Bounded section
 BS     =      82.00  AS     =     596.96  QA     =      48.52  ICHREG= 1
 HA     =       7.28  HD     =       7.28
 UA     =       .081  F      =       .016  USTAR = .3658E-02
 UW     =      1.700  UWSTAR= .1854E-02
 Uniform density environment
 STRCND=  U           RHOAM = 1015.8300

DISCHARGE PARAMETERS (metric units)
 BANK   =  RIGHT      DISTB =        .50  Configuration: protruding_discharge
 SIGMA =     90.00    HD0    =        .46  SLOPE =      42.00
 Circular discharge pipe:
 D0     =       .457  A0     =       .101
 Dimensions of equivalent rectangular discharge:
 B0     =       .359  H0     =       .457  A0     = .1640E+00  AR     =      1.273
```

| | | | | | | |
|---|---|---|---|---|---|---|
| 118.16 | -.50 | 0.00 | 8.4 | .119E+02 | .05 | 66.93 |
| 121.05 | -.50 | 0.00 | 8.6 | .116E+02 | .05 | 67.87 |
| 123.95 | -.50 | 0.00 | 8.8 | .113E+02 | .05 | 68.79 |
| 126.85 | -.50 | 0.00 | 9.1 | .110E+02 | .06 | 69.71 |
| 129.74 | -.50 | 0.00 | 9.3 | .107E+02 | .06 | 70.62 |
| 132.64 | -.50 | 0.00 | 9.5 | .105E+02 | .06 | 71.53 |
| 135.54 | -.50 | 0.00 | 9.8 | .102E+02 | .06 | 72.43 |
| 138.43 | -.50 | 0.00 | 10.0 | .998E+01 | .06 | 73.33 |
| 141.33 | -.50 | 0.00 | 10.3 | .973E+01 | .06 | 74.22 |
| 144.23 | -.50 | 0.00 | 10.5 | .950E+01 | .06 | 75.10 |
| 147.12 | -.50 | 0.00 | 10.8 | .927E+01 | .06 | 75.98 |
| 150.02 | -.50 | 0.00 | 11.1 | .904E+01 | .06 | 76.86 |
| 152.91 | -.50 | 0.00 | 11.3 | .883E+01 | .06 | 77.73 |
| 155.81 | -.50 | 0.00 | 11.6 | .861E+01 | .06 | 78.59 |
| 158.71 | -.50 | 0.00 | 11.9 | .841E+01 | .06 | 79.45 |
| 161.60 | -.50 | 0.00 | 12.2 | .821E+01 | .06 | 80.31 |
| 164.50 | -.50 | 0.00 | 12.5 | .801E+01 | .07 | 81.16 |
| 167.40 | -.50 | 0.00 | 12.8 | .783E+01 | .07 | 82.00 |

Cumulative travel time = 3256. sec
Plume is LATERALLY FULLY MIXED at the end of the buoyant spreading regime.

END OF MOD341: BUOYANT AMBIENT SPREADING

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BEGIN MOD361: PASSIVE AMBIENT MIXING IN UNIFORM AMBIENT

Vertical diffusivity (initial value) = .617E-02 m^2/s
Horizontal diffusivity (initial value) = .771E-02 m^2/s

Profile definitions:
BV = Gaussian s.d.*sqrt(pi/2) (46%) thickness, measured vertically
= or equal to water depth, if fully mixed
BH = Gaussian s.d.*sqrt(pi/2) (46%) half-width,
measured horizontally in Y-direction
S = hydrodynamic centerline dilution
C = centerline concentration (includes reaction effects, if any)

Plume Stage 2 (bank attached):

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 167.40 | -.50 | 0.00 | 12.8 | .783E+01 | .07 | 82.00 |
| 180.45 | -.50 | 0.00 | 12.8 | .783E+01 | .07 | 82.00 |
| 193.50 | -.50 | 0.00 | 12.8 | .783E+01 | .07 | 82.00 |
| 206.55 | -.50 | 0.00 | 12.8 | .783E+01 | .07 | 82.00 |
| 219.61 | -.50 | 0.00 | 12.8 | .783E+01 | .07 | 82.00 |
| 232.66 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 245.71 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 258.76 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 271.81 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 284.87 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 297.92 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 310.97 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 324.02 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 337.07 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 350.13 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 363.18 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 376.23 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 389.28 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 402.33 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 415.39 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 428.44 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 441.49 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 454.54 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 467.59 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 480.65 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 493.70 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 506.75 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |
| 519.80 | -.50 | 0.00 | 12.8 | .782E+01 | .07 | 82.00 |

| | | | | | | |
|---|---|---|---|---|---|---|
| -3.97 | .00 | 0.00 | 1.8 | .542E+02 | .05 | 13.02 |
| -.65 | .00 | 0.00 | 1.7 | .587E+02 | .06 | 15.70 |
| 2.66 | .00 | 0.00 | 1.8 | .553E+02 | .06 | 17.99 |
| 5.98 | .00 | 0.00 | 2.2 | .453E+02 | .06 | 20.01 |
| 9.30 | .00 | 0.00 | 2.7 | .366E+02 | .06 | 21.85 |
| 12.62 | .00 | 0.00 | 3.2 | .312E+02 | .06 | 23.55 |
| 15.94 | .00 | 0.00 | 3.5 | .284E+02 | .06 | 25.13 |
| 19.25 | .00 | 0.00 | 3.7 | .271E+02 | .06 | 26.61 |
| 22.57 | .00 | 0.00 | 3.8 | .263E+02 | .06 | 28.02 |

Cumulative travel time = 439. sec

END OF MOD331: UPSTREAM INTRUDING PLUME

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** End of NEAR-FIELD REGION (NFR) **

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BEGIN MOD341: BUOYANT AMBIENT SPREADING

Plume is ATTACHED to RIGHT bank/shore.
Plume width is now determined from RIGHT bank/shore.

Profile definitions:
BV = top-hat thickness,measured vertically
BH = top-hat half-width, measured horizontally from bank/shoreline
S = hydrodynamic average (bulk) dilution
C = average (bulk) concentration (includes reaction effects, if any)

Plume Stage 2 (bank attached):

| X | Y | Z | S | C | BV | BH |
|---|---|---|---|---|---|---|
| 22.57 | -.50 | 0.00 | 3.8 | .263E+02 | .06 | 28.01 |
| 25.47 | -.50 | 0.00 | 3.9 | .256E+02 | .06 | 29.68 |
| 28.37 | -.50 | 0.00 | 4.0 | .250E+02 | .05 | 31.29 |
| 31.26 | -.50 | 0.00 | 4.1 | .245E+02 | .05 | 32.83 |
| 34.16 | -.50 | 0.00 | 4.2 | .239E+02 | .05 | 34.33 |
| 37.05 | -.50 | 0.00 | 4.3 | .234E+02 | .05 | 35.78 |
| 39.95 | -.50 | 0.00 | 4.4 | .228E+02 | .05 | 37.19 |
| 42.85 | -.50 | 0.00 | 4.5 | .223E+02 | .05 | 38.57 |
| 45.74 | -.50 | 0.00 | 4.6 | .218E+02 | .05 | 39.90 |
| 48.64 | -.50 | 0.00 | 4.7 | .213E+02 | .05 | 41.21 |
| 51.54 | -.50 | 0.00 | 4.8 | .208E+02 | .05 | 42.48 |
| 54.43 | -.50 | 0.00 | 4.9 | .204E+02 | .05 | 43.73 |
| 57.33 | -.50 | 0.00 | 5.0 | .199E+02 | .05 | 44.96 |
| 60.23 | -.50 | 0.00 | 5.1 | .194E+02 | .05 | 46.16 |

** REGULATORY MIXING ZONE BOUNDARY **
In this prediction interval the plume distance meets or exceeds
the regulatory value = 62.00 m.
This is the extent of the REGULATORY MIXING ZONE.

| | | | | | | |
|---|---|---|---|---|---|---|
| 63.12 | -.50 | 0.00 | 5.3 | .190E+02 | .05 | 47.33 |
| 66.02 | -.50 | 0.00 | 5.4 | .185E+02 | .05 | 48.49 |
| 68.92 | -.50 | 0.00 | 5.5 | .181E+02 | .05 | 49.63 |
| 71.81 | -.50 | 0.00 | 5.7 | .177E+02 | .05 | 50.75 |
| 74.71 | -.50 | 0.00 | 5.8 | .172E+02 | .05 | 51.85 |
| 77.61 | -.50 | 0.00 | 5.9 | .168E+02 | .05 | 52.94 |
| 80.50 | -.50 | 0.00 | 6.1 | .164E+02 | .05 | 54.01 |
| 83.40 | -.50 | 0.00 | 6.2 | .160E+02 | .05 | 55.07 |
| 86.30 | -.50 | 0.00 | 6.4 | .156E+02 | .05 | 56.12 |
| 89.19 | -.50 | 0.00 | 6.6 | .153E+02 | .05 | 57.15 |
| 92.09 | -.50 | 0.00 | 6.7 | .149E+02 | .05 | 58.18 |
| 94.98 | -.50 | 0.00 | 6.9 | .145E+02 | .05 | 59.19 |
| 97.88 | -.50 | 0.00 | 7.1 | .142E+02 | .05 | 60.19 |
| 100.78 | -.50 | 0.00 | 7.2 | .138E+02 | .05 | 61.18 |
| 103.67 | -.50 | 0.00 | 7.4 | .135E+02 | .05 | 62.16 |
| 106.57 | -.50 | 0.00 | 7.6 | .131E+02 | .05 | 63.13 |
| 109.47 | -.50 | 0.00 | 7.8 | .128E+02 | .05 | 64.09 |
| 112.36 | -.50 | 0.00 | 8.0 | .125E+02 | .05 | 65.05 |
| 115.26 | -.50 | 0.00 | 8.2 | .122E+02 | .05 | 65.99 |

```
REGMZ  =   1
REGSPC=   1          XREG   =     62.00  WREG   =        .00  AREG   =        .00
XINT   =       820.00  XMAX   =    820.00

X-Y-Z COORDINATE SYSTEM:
    ORIGIN is located at the WATER SURFACE and at center of discharge
      channel/outlet:        .50 m  from the RIGHT bank/shore.
    X-axis points downstream
    Y-axis points to left as seen by an observer looking downstream
    Z-axis points vertically upward (in CORMIX3, all values Z = 0.00)
NSTEP = 50 display intervals per module
-----------------------------------------------------------------------------

        TRJBUO     TRJATT     TRJBND     TRJNBY     TRJCOR     DILCOR
C        3.848      1.000       .991       .991      3.812      1.000
-----------------------------------------------------------------------------
BEGIN MOD301: DISCHARGE MODULE

 Efflux conditions:
       X         Y        Z         S        C         BV        BH
      .00       .00      0.00      1.0   .100E+03     .28       .18

END OF MOD301: DISCHARGE MODULE
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD302: ZONE OF FLOW ESTABLISHMENT

  Control volume inflow:
       X         Y        Z         S        C         BV        BH
      .00       .00      0.00      1.0   .100E+03     .28       .18

 Profile definitions:
   BV = Gaussian 1/e (37%) vertical thickness
   BH = Gaussian 1/e (37%) horizontal half-width, normal to trajectory
   S  = hydrodynamic centerline dilution
   C  = centerline concentration (includes reaction effects, if any)

  Control volume outflow:
       X         Y        Z         S        C         BV        BH
      .08       .35      0.00      1.7   .588E+02     .58       .43
 Cumulative travel time =          2. sec

END OF MOD302: ZONE OF FLOW ESTABLISHMENT
-----------------------------------------------------------------------------
-----------------------------------------------------------------------------
BEGIN MOD331: UPSTREAM INTRUDING PLUME
```

# APPENDIX C

APPENDIX C

# City of Toledo

LINCOLN COUNTY, OREGON

## Inflow and Infiltration Study

***May 2011***

TOLEDO WELCOMES YOU

REGISTERED PROFESSIONAL
ENGINEER
56560PE
OREGON
JULY 9, 2001
JAMES G. PALLO
EXPIRATION DATE: 12/31/12

## Table of Contents

**APPENDICES**

Section
1

# 1.0 Executive Summary

## 1.1 Background

The City of Toledo has historically struggled with high levels of inflow and infiltration (I/I) in their wastewater system. This is most evident during the winter months when stormy conditions cause flows in the system to rise dramatically as rain and groundwater enters the sewer system.

Though not currently under a mandated order (MAO) from DEQ, the City does have a history of overflows and untreated or partially treated sewage spills into the river. The treatment plant regularly bypasses partially treated wastewater that exceeds the capacity of the facility. The current Wastewater Master Plan (Clearwater 1995), seeking to reduce these bypasses, recommended improvements to the City pump stations and treatment plant. Those improvements, completed in the late 1990's, were calculated to be a more cost effective method to reduce the sewage spills than pursuing I/I reduction.

While substantial improvement has been seen in spill reduction from the treatment and pumping upgrades, the City still experiences high I/I levels that will continue to increase as the collection system ages. Due to the historic nature of the City, the average age of the collection system is higher than many younger cities. Therefore, an aggressive I/I program will require sizeable repairs throughout the system.

The last concerted effort to reduce I/I was completed in the early 1990's, and involved extensively replacing some of the worst system components with new pipe and manholes. Reportedly, this repair work was successful though the magnitude of the deficiencies left many further components still in need of repair or replacement.

During the summer of 2009 and winter of 2009-2010, the City contracted with Civil West Engineering Services to complete a detailed round of smoke testing and flowmapping of the complete sanitary sewer collection system. The projects were a success as many leaks were located, mapped, and categorized. Follow-up efforts by the City to correct residential-owned deficiencies has been successful, with a reported high level of resident compliance and measured flows into the treatment plant reduced.

After completion of these I/I field surveys the City authorized a television inspection survey and this I/I study to complete further analysis of I/I issues. This report will develop a capital improvement plan with the goal of undertaking cost effective projects to reduce the amount of I/I in the collection system. Reduction of I/I in Toledo will extend the useful life of the collection system, pump stations, and treatment plant saving sewer customers money. It will also help the City avoid sewage spills that may result in stiff penalties and fines from DEQ.

## 1.2 Overview of Results from Surveys

Three investigative surveys were provided by Civil West to pinpoint I/I sources within the system. The Smoke Testing Survey discovered nearly 200 individual deficiencies in the collection system, the Flow Mapping Survey discovered 8 large pipe and 17 manhole deficiencies, and the Television Inspection Survey discovered dozens of mainline pipe and lateral deficiencies. The Television Inspection Survey inspected approximately 10% of the gravity sewer pipelines.

### 1.2.1 Recommended Improvement Projects

Analysis of the three authorized studies during this I/I report facilitated the creation of many individual improvement projects. In summary those projects consist of:

- 5 Complete Pipe Replacement Projects
- 5 Pipe Lining Projects
- 2 Bursting Projects
- 1 Pipe Patching Project
- 2 Manhole Rehabilitation Projects
- 1 In-Pipe Repair Project

Pipe replacement is the most invasive type of repair work, where a new trench must be dug and a plan to maintain or bypass sewer service during construction implemented. Lining, bursting, and patching projects can often be done in several hours after preparation work. They are non-invasive and result in little ground disturbance, short interruptions to sewage flows, and are generally less costly. Consequently non-invasive projects were preferred when judged feasible.

Approximately 6000 feet of pipe and nearly 30 manholes have been recommended for repair or replacement. As such, not all the suspected deficiencies have been fully investigated making it likely that numerous undiscovered deficiencies remain in the system.

This first round of evaluation was aimed at locating and identifying “low-hanging fruit” or problems that can be corrected in a cost effective way resulting in a strong cost/benefit approach. This should not be considered a “final” I/I study.

## 1.3 Summary of Capital Improvement Plan and Funding

A total combination of all the projects recommended in this study resulted in a cost in today’s dollar of **$1,436,675.** It is not feasible for any public utility operator to complete all of their needed improvements immediately following an analysis. Therefore to better organize rehabilitation efforts by the City, the various projects have been prioritized and ranked to allow the City to manage their resources and get the greatest benefit for each dollar invested in I/I rehabilitation.

The Capital Improvement Plan (CIP) has been broken into four priority levels, with lower numbers reflecting the most urgent repairs.

- **Priority 1**, projects which need immediate repairs with large deficiencies and extreme I/I.
    - **Total Repairs $380,935**
- **Priority 2**, projects which need repair over the next few years. Deficiencies are nearly as serious as Priority 1 but may be delayed to attain funding.
    - **Total Repairs $565,400**
- **Priority 3**, projects with less systemic deficiencies and more isolated I/I points. Repair is suggested before the next 5-6 years.
    - **Total Repairs $350,260**
- **Priority 4**, projects mainly needing point repairs or with minor deficiencies that were not observed contributing substantial I/I to the collections system.
    - **Total Repairs $140,080**

It is anticipated that the City will pursue funding assistance in completing the more urgent projects and, potentially, all of the projects. Along with sanitary sewer repairs, the City is facing sizeable repairs to their drinking water system. The combination of these costs suggests funding will need to come from a variety of sources, including ratepayers, and public funding agencies.

At a minimum, the City should seek to address the Priority 1 & 2 repairs while actively monitoring the collection system for other serious problems.

# Background and Need

*Section*

**2**

## 2.0 Background

The City of Toledo owns and maintains a wastewater conveyance system that includes the following:

- A sanitary sewer system that includes a wastewater collection system, several pumping stations, a treatment plant, and a river outfall for treated effluent.
- Original concrete piping built in 1920's
- New PVC piping installed in the early 90's.
- Various repair patches of ABS and PVC pipe and some lined pipe sections.

The City has completed planning efforts and intends to undertake improvements to their water and wastewater infrastructure in response to development pressures and the need to upgrade and update aging infrastructure components.

The purpose of this study is to evaluate specific deficiencies within the wastewater collection system and to develop a rehabilitation plan with specific recommendations to enable the City to reduce their overall I/I.

### 2.0.1 Summary of Previous I & I reduction efforts

The City authorized this I/I report and associated surveys. The following provides a summary of the previous planning efforts which, at least in part, addressed the I/I problem.

1. Wastewater Facilities Plan: Completed in December 1993 by Clearwater Engineering Corporation, the current Facilities Plan includes recommendations for improvements in the collection system and the treatment facilities.

2. Wastewater Master Plan: The City's water master plan was completed in August of 1995 by Clearwater Engineering Corporation. The Plan continues the recommendations made in the 1993 Facilities Plan and recommends a schedule and funding sources for completing them.

Approximately 20 years ago, from 1990-1991, significant I/I repairs were made to the collection system, including 12,000 feet of sewer mainline, 3200 feet of sewer trunk, 60 manholes, and 200 service laterals. These repairs were seen as successful by reducing storm overflows caused by a 3-year rain event (A 3-year rain event is equal to a 24 hour period of rainfall of such volume that it occurs, statistically, once every 3 years). Later improvements to the treatment and pumping system were developed to reduce overflows for up to 5-year rain event.

## 2.2 Need for This Report

I/I is a common problem in Western Oregon where wet weather persists through much of the year and many cities have aged and leaky collection systems. Winter rainfall makes its way into wastewater facilities from the surface by way of improperly connected drains and cracks in the ground, or underground through broken pipes, joints, and manholes when the water table is high. This additional water creates an unnecessary cost burden on the entire treatment system as it requires larger pipes, pump stations and treatment facilities.

The City has addressed its I/I problems in the recent past by upsizing facilities to handle the high flows and only repairing pipelines when it makes financial sense. In past studies it was determined that it was more cost effective to treat the excess I/I problem than to rehabilitate the conveyance system. Extensive upgrades were completed to the wastewater treatment plant to eliminate overflows caused by heavy rainfall.

Even with threats of overflows reduced, the City must maintain its current system. The original concrete pipes and manholes continue to deteriorate, adding greater flows to the system. As the City grows and expands its system it continues to incur pumping and treating costs to handle flows which should be channeled into the stormwater system. The current NPDES permit, which allows the wastewater plant to discharge to the Yaquina River, is up for renewal this November and I/I reduction efforts will likely be required as part of that permit renewal.

Additionally, the City has made no concerted effort to target and reduce I/I in 20 years. With an already aging system, 20 years is a long period of time of unchecked deterioration.

## *2.3 Report Organization*

The following sections comprise this City of Toledo I/I Report as presently constituted:

- **Section 1 – Executive Summary.** This section provides a brief overview and summary of the I/I reduction strategy and is intended to provide the reader with the important facts and findings contained in the overall plan.
- **Section 2 – Background and Need.** This section provides information on the background of the issues and describes the need for the report so that readers understand why a reduction of I/I is important.
- **Section 3 – Summary of Smoke Testing Survey.** This section describes the methodology and results of the first phase of investigating sources of inflow into the conveyance system. It explains to the reader where likely sources of inflow exist and what should be done about them.
- **Section 4 – Summary of Flow Mapping Survey.** This section describes the methodology and results of night time flow mapping performed throughout the city. It provides the locations where excess water is infiltrating into damaged manholes and piping.
- **Section 5 – Summary of Television Survey.** This section will serve as a summary of the all the video footage taken from within the collection system. This includes details about what types of deficiencies were found, where they exist, and the most suitable repair type to use.
- **Section 6 – Rehabilitation Methods.** Based upon the results of the earlier sections, this section describes alternative repair methods available to the City along with their strengths and weaknesses.
- **Section 7 – Improvement Projects.** This section builds upon the data from Sections 5 and 6 to develop an organized set of projects to repair the collection system. It includes the suggested repair method and an estimated cost to complete the project.
- **Section 8 – Capital Improvement Plan and Financing Options.** Based on the analysis in Section 7, this section will provide specific recommendations and direction on the implementation and funding strategy for the planned projects.
- **Appendix.** The Appendix includes information that is referenced in this study but is not included in the referenced planning documents.

Section
3

## 3.0 Summary of Smoke Testing Survey

### 3.1 Smoke Testing Method

Smoke testing is an engineering-surveying tool used to locate, identify, and classify potential inflow/infiltration sources in a wastewater collection system. Simply put, smoke testing involves pumping large volumes of smoke into the collection system through an open manhole. This is accomplished using a blower that sits directly over a manhole. Smoke is generated through the use of "smoke bombs" or other means.

The smoke travels down the piping under a small amount of positive pressure created by the blower. The smoke filled air seeks locations to escape the piping system. This may include "escape points" that are normal and acceptable such as:

- Roof vent pipes (plumbing stacks)
- Manhole lid holes

Other observed points where smoke escapes may be indicative of leaks in the system. This may include:

- Leaks in the piping and fissures leading to the ground surface
- Open cleanouts
- Cross-connections to the storm drainage system
- Downspouts on buildings
- And others.

**Figure 3.1 Smoke Testing**

It is the negative escape points or "smoke return" locations that the smoke test survey is intended to locate. "Smoke return" locations often indicate where inflow from rainfall is entering the system and occasionally reveal infiltration sources as well.

### 3.2 Smoke Testing Results

The smoke testing effort identified nearly 200 individual deficiencies throughout the wastewater conveyance system. As is often the case, many of the deficiencies are easily correctable occurrences located on residential properties. These include missing cleanout caps or cleanouts used as catch basins, gutter downspouts connected to the sewer system, and obvious plumbing code violations.

Initial results of the Smoke Testing Survey were presented in the Systemwide Sanitary Smoke Testing Executive Summary (Civil West 2009). The initial results were studied along with results of the Television Survey to more accurately determine the deficiency class of each smoke return. (see Appendix C). A summary of the updated results is:

- 51 Broken lateral pipes
- 40 Broken mainline pipe locations
- 13 Catch basins tied into sewer
- 3 Private residential catch basins tied to sewer
- 6 Gutter downspouts tied to sewer
- 4 Apparent plumbing code violations
- 36 cracked or leaking manholes
- 34 Broken or uncapped private cleanouts

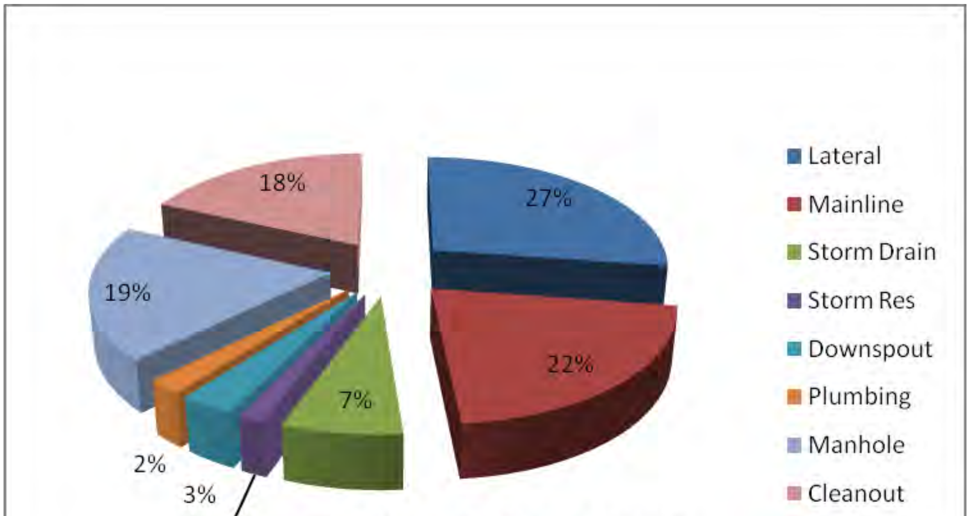


Figure 3.2 Smoke Testing Results

Maps provided in the Appendix C show the detailed locations of each smoke return in the Smoke Testing Survey. The City was provided with sample letters to notify residents of deficiencies on their property contributing to I/I that can be corrected and followed up with this recommendation. The City promptly utilized the letters and made significant progress in eliminating the sources of inflow.

There are also many more difficult deficiencies to repair within the conveyance system. These include broken pipes, displaced pipe gaskets, municipal storm drains connected to the sewer, and cracked or leaking manholes. Broken pipes may either be larger mainline sewers operated by the City's Public Works department or service laterals on private property.

For purposes of further investigation on the part of the City, it is difficult and costly to inspect each of the 51 damaged service laterals unless they are selected for repairs or observed in other surveys to be defective. For information about the location of laterals consult Appendix C and the Systemwide Sanitary Smoke Testing Executive Summary (provided to the City by Civil West Engineering Services after completion of the Smoke Testing Survey). Deficient manholes can be visually inspected by City staff and are categorized in Appendix B.

Table 3.2 lists the remaining smoke returns which likely can be attributed to deficiencies with the City's sewer piping. They have been categorized into two groups, one group showing a significant pipe failure and the other group where the deficiencies are small enough to warrant a spot repair. This result, combined with the results for the Flow Mapping Survey and Television Survey, will form the basis for repair recommendations in the Improvement Plan in Section 7.

**TABLE 3.2**

| Pipe segments showing significant deficiencies through smoketesting<br>Pipe Segment | |
|---|---|
| | **Pipe Segment** |
| Long section with multiple breaks | K11 to K16 |
| Several locations of smoke coming from ground | F23 to F26 |
| Many locations with smoke emitting along street | B14 to B22 |
| Smoke arising from field in several spots | B38 to B40 |
| Smoke from ground following pipeline | I69 to I74 |
| Many cracks in streets emitting smoke | I69 to I72 |
| Ditch line smoking | N3 to N4 |
| Large hole in line | D9 to D11 |
| Smoke coming from ground around pipeline | F18 to F20 |
| Water Meter emitting smoke | K28 to K29 |
| Smoke appearing in fields around pipe | H28 to H29 |
| Large holes in ground emitting smoke | K37 to K38 |
| **Pipe segments showing some deficiencies through smoketesting** | |
| | **Pipe Segment** |
| Smoke observed in bushes | B70 to B71 |
| Road shoulder smoking | O6 to O7 |
| Section of pipe smoking south of manhole C6 | C5 to C6 |
| Smoke in bushes could be buried manhole or void | C9 to C13 |
| Smoke coming from trees | F17 to F27 |
| Several locations of smoke coming from ground | F23 to F24 |
| Smoke near both manholes | F50 to F51 |
| Smoke from ground around construction site | E2 to E3 |
| Smoke from retaining wall | I18 to I19 |
| Several cracks in pavement emitting smoke | I28 to I29 |
| 3 locations with smoke from ground | I23 to I84 |
| Smoke by manhole and to the south | K23 to K26 |
| Ground emitting smoke along driveway | K29 to K28 |
| Smoke coming from field along pipeline | M13 to M18 |
| Holes in the ground over what appears to be mainline | I46 to I47 |

## 3.3 Smoke Testing Conclusions

Feedback from the City Public Works Department reports a high degree of compliance resulting from the repair letters delivered to residents. Reductions in the overall flows at the wastewater treatment plant have been noted and are, presumably, due to early successes in I/I reduction. Once the "low hanging fruit" deficiencies are repaired, such as those addressed within the notification letters, the more costly and difficult to repair deficiencies must be remedied. The remaining repairs include leaking manholes, catch basin separation and broken underground pipes.

Manhole problems have been listed and indexed in the Appendix by manhole number and included in the repair project section. Many of the manholes have been fully or partially repaired by the City based upon the smoketesting results. Unless a sizeable structural collapse has occurred, manholes typically can be reinforced and rehabilitated to good condition.

Catch basin connections can be found using the smoketesting report. Only a relatively small number of catch basins were found with potential tie-ins to the sanitary sewer. We estimate that connections to the sanitary sewer system are most likely due to underground voids between the storm and sanitary system based upon where the smoke returns were seen and subsequent television inspection. In other words, "connections" between the storm and sanitary sewer are often due to cracked or broken pipes being in close proximity to each other and not necessarily a result of direct connections.

Municipal catch basins with a smoke return can be indicative of either an active tie-in to the sewer system or faulty underground conditions that allow mixing of sewer and storm water. These were not specifically checked for in future surveys as flow mapping was conducted during rainless nights and the television surveys were used to investigate infiltration. The City should conduct dye testing where a fluorescent non-toxic dye is poured into the catch basins while inspecting nearby sewer pipes with a camera. If the catch basins are actively connected to the sewer network the dye will enter through a lateral. If the dye enters through pipe joints or manhole rings it will be evident there is an underground void connecting the two systems.

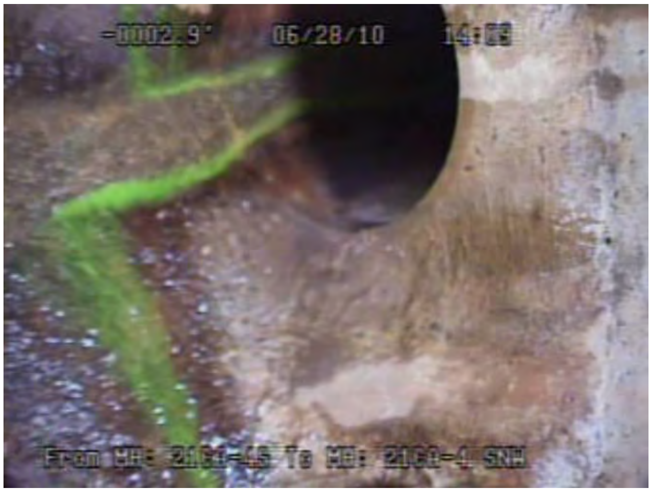


**Figure 3.3 Fluorescent Tracer Dye**

Broken underground pipes can be separated into laterals and mainline breaks. Mainline breaks can be found through television inspection and repaired by the city. Those marked as such in the Smoke Testing Survey were televised.

Lateral breaks are more complicated because the lateral piping is shared between the residential owner and the City. Some lateral breaks are visible during televising if they are located near the mainline. If the breaks are located on private property or towards the cleanout, a separate television inspection must be done on each lateral. Unusual flows from laterals are documented while televising the mainline and can be helpful in determining problems with the lateral that cannot be observed directly.

Typically any sewer repairs that replace the sewer mainline will include replacing the lateral up to the property line. This may reduce I/I but the City must coordinate a plan with property owners if they wish to completely stop I/I within a lateral connection.

Pipe segments that show evidence of problems due to underground breakage or leaks include those listed in Table 3.2

Section
4

## 4.0 Summary of Flow Mapping Survey

This section describes in detail how flow mapping is accomplished, what it can tell us about the collection system, and what the results of the survey indicate.

### *4.1 Flow Mapping Method*

Flow mapping is accomplished through the use of a flow meter (commonly called a "Flow Poke") that can be quickly and easily inserted into a pipeline through a manhole. The meter allows for an instantaneous flow measurement in gallons per minute of sewage flow through a sewer pipe. Another flow reading can then be made at an upstream manhole that allows a comparison between the two manholes. If it is found that there is more flow in the downstream manhole than the upstream manhole, it can be concluded that an infiltration problem exists between the two manholes.

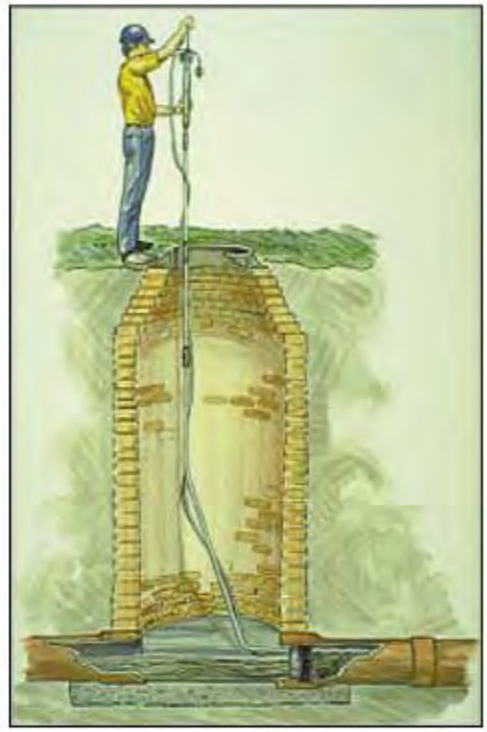


**Figure 4.1 Flow Poke**

The flow information is drafted onto a map of the system to show the location and amounts of flows in the system at the time the measurements were made. This allows the engineer to review the entire system and determine where additional investigation is warranted. Flow mapping is completed during the mid-night hours (11 pm to 6 am) when the vast majority of flow in the collection system is I/I as domestic flows are significantly reduced after 10 pm. The goal is to measure the consistent flows generating from underground leaks while not measuring the widely varying flows coming from sinks, toilets and other residential uses.

The team conducting the flowmapping consists of one person holding the flow poke into the manhole and the other taking the flow readings. The team also inspects the manhole at the insertion site for condition and visible signs of leaks. Flow mapping begins at the bottom of a sanitary drainage basin and proceeds up the basin by taking measurements at each sewer inlet to the manholes. If the flow is found insignificant no further investigation is required. If high flows are recorded the team continues to "follow" the flow by proceeding upstream through each manhole until that flow too becomes insignificant. This process creates a fast and effective method to discover sizeable problems throughout the collection system..

### *4.2 Flow Mapping Results*

The Flow Mapping Survey mapped the complete collection system within the area operated by the City. Flows deemed significant were followed and measured. Negligible or zero flows were marked in the engineering field books and no further investigation is required. Table 4.2 lists the all the major areas of concern where unaccounted flows were found.

**TABLE 4.2**

| Manholes | Street Location | Indicator | Length |
|---|---|---|---|
| B29 TO B31 | N Nye St, just North of NW 15th | 20 gpm potential infiltration | 440 |
| B12 TO B22 | NW 12th St from Spruce to Arcadia St | 7 gpm potential infiltration | 640 |
| B1 TO B9 | NW 11th and Meadow Lane | 18 gpm potential infiltration | 120 |
| C1 TO C21 | Lincoln Way and NW Westwood | >10gpm potential infiltration | 180 |
| D4 TO F8 | Business 20 across from Police | 20 gpm potential infiltration, large manhole leaks observed | 550 |
| I4 TO I34 | E Graham | 20 gpm potential infiltration | 570 |
| I26 TO I29 | SE Alder between SE 2nd and 1st | >15 gpm potential infiltration | 370 |
| F8 TO B1 | A St North of Business 20 | Multiple potential infiltration points | 1730 |

Additional sections of the collection system were found to contain possible infiltration flows. However, these flows were small enough to be within the margin of error of the equipment or typical nightly domestic flow. The practical limitation of short duration flow mapping is that it works best at finding large deficiencies and helps to identify where to conduct television surveys.

Manholes discovered with visible leaking during the Flow Mapping Survey have been included in the same Table (7.2.15A) that those from the smoketesting report have been listed in. A follow up investigation performed during January 2011 further refined the results based upon City repairs and confirmed locations. Deficiencies seen in flow mapping tend to be seen at the deeper levels and joints of the manhole, when water table is high, whereas those deficiencies found from smoke testing can include deficiencies at the top of the manhole and cracks under the rim.

It was noted that the City has already undertaken good measures to stop inflow into manholes such as providing many sloped areas with rain shielding inserts and 2-hole lids. Many of the covers in high traffic areas were found to be bolted down which limited some investigation possibilities.

## *4.3 Flow Mapping Conclusion*

Several very significant leaks were found through the use of flow mapping, in both sewer pipe and through sanitary manholes. Each of these locations were recommended for television inspection and reviewed further in this study. Detailed results can be seen in the maps included in the Appendix.

Flow mapping should be repeated after repairs to the system are complete to help calculate the effectiveness of those repairs as well as to identify new deficiencies. Another useful tool is to conduct a manhole inspection during high groundwater months. Because the City contains a proportionally high number of manholes, and flowmapping only illuminates heavily leaking manholes, it would be useful for collection systems crews to keep a log of manhole leaks and inspections. Manhole repairs are a relatively inexpensive source of I/I reduction due to their accessibility.

Section
5

# 5.0 Summary of Television Survey

## 5.1 Television Survey Method

This section describes in detail how cleaning and televising is performed

Television inspection is a tool that, when combined with smoke testing and flow mapping, can help determine what rehabilitation measure should be taken within a collection system. While smoke testing and flow mapping reveal potential problems within a system, a television survey allows the Engineer to see directly into the pipe and pinpoint infiltration sources and pipe cracks and breaks.

**Figure 5.1.1 Jetter Truck**

The inspection itself is a two part process. First, the pipe and manholes must be cleaned free of all dirt, grease, rock and other debris. This is accomplished by the use of a "jetter truck." The jetter truck contains a powerful pump that connects to a cleaning nozzle on a hose reel. The hose is inserted into a manhole as the nozzle jets water back towards the hose and propels itself down the pipe through water pressure. Once the nozzle reaches the next manhole the operator retracts the hose slowly and pulls the debris back towards the insertion manhole. A large vacuum system mounted on the truck removes the debris through the manhole into a storage tank. This process is repeated until the pipe and manhole are clean. The jetter truck separates the water from the debris and discharges the water back into the conveyance system and discharges the debris at an approved site.

**Figure 5.1.2 Televising Camera**

Televising is the second part of the process. A robotic camera is lowered into the manhole and remotely controlled to crawl through the pipe. The camera is tethered to the truck by a cable which provides power and communications between the camera and truck as well as providing a tool for measuring distances.. The camera provides a light source and moves along the pipe recording important features such as sewer lateral locations, pipe joints, and abnormalities. The operator maintains a log of the inspection process and digitally records the investigation. When complete, the logs and video are delivered to the engineer for review.

## 5.2 Television Survey Results

The final Television Survey cataloged 60 individual pipe segments totaling 10,200 feet of the approximately 98,800 feet of installed sewer pipe. A segment shall be defined as a continuous pipeline beginning at a manhole and ending at another manhole or sewer cleanout. Not all of these segments were inspected in their entirety due to blockages or pipe offsets preventing further camera travel.

Observation of the video results reveals the following:

- 25 Segments are in average or better condition without any need for further work.
- 4 Segments need further investigation
- 5 Segments are in need of minor repairs that may be spot repairs
- 8 Segments require more major repairs or replacement but are not causing large problems yet
- 15 Segments have major damage throughout the pipe and should be repaired soon
- 3 Segments are near imminent failure

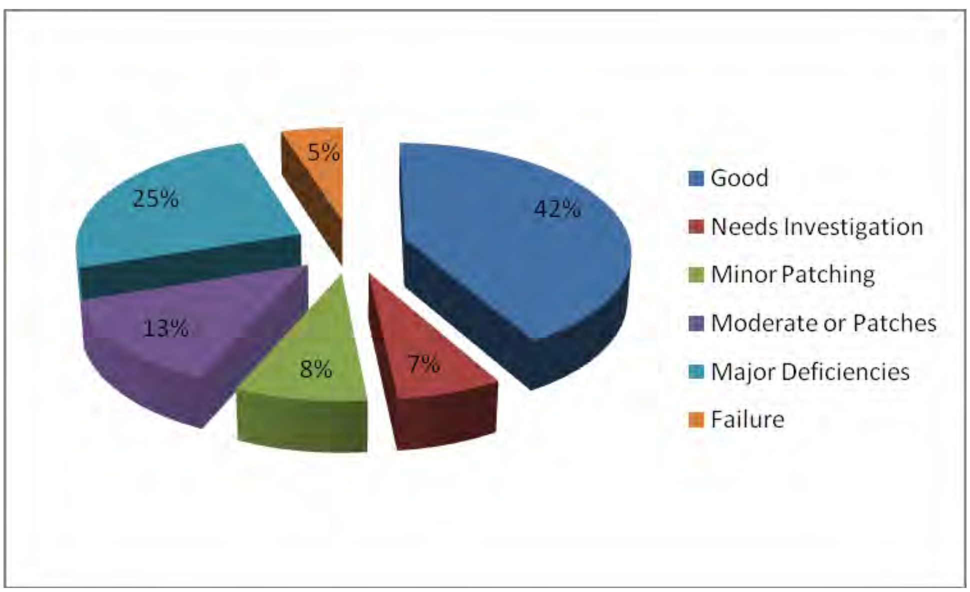



**Figure 5.2 Televising Results**

Overall, PVC and clay tile pipes are in good condition while the concrete pipe is typically either failing, near failure, or the pipe appears old and worn. Where liners are installed in the pipes, the liners are in good condition and providing good service. Short pipe patches are also performing well, though it can be observed that the pipe adjacent to them is now deteriorating and that they are a short term solution.

Several locations were completely obstructed and the pipe was not fully investigated. These items are noted in the report pages in Appendix (A). These obstructions are typically heavy root intrusion that the cleaning nozzle could not dislodge or protruding laterals blocking passage of large items, including the camera. One pipe in the downtown area contained large asphalt or concrete pieces making television inspection impossible.

The televising contractor noted that Toledo's sewer system contained higher than average amounts of sediment build up, specifically grit and gravel accumulations along some of the main trunk lines. The indication would be that the pipes require more regular cleaning intervals. Grease buildup that was seen inside the pipe was typical and not excessive.

## 5.3 Television Survey Conclusion

Areas with deficiencies observed during televising have been categorized in the previous section. Improvement projects have been developed to address each deficiency. Several of the low lying pipe segments were difficult to televise due to large "bellies" in the pipe. Incomplete information was gathered in these "bellied" pipes as the camera was submerged and the pipe walls and joints were not visible on camera. The large bellies are not acceptable in the pipe as they reduce the carrying capacity of the pipe and result in buildup of debris and detention time of waste. These pipes are recommended for replacement.

Many of the laterals were observed to be leaking heavily and were included in rehabilitation projects. Typically, the cause of the leak was directly observable by camera from the mainline pipe or at the lateral connection. Any additional lateral televising we determined as necessary was included into the overall lateral replacement price of the rehabilitation projects in Section 7.

Several pipes recommended for inspection were unable to be televised while remaining within the budget allocated for the City. These pipes were those difficult to access and require portable type televising equipment. We recommend that the City set aside budget to televise these lines as well as other difficult to access areas that the Public Works department suspects have deficiencies.

The following pipes should be scheduled for inspection as soon as possible:

**TABLE 5.3 – PIPES SEGMENTS REMAINING TO BE TELEVISED**

| Pipe Segment (s) | Street Location | Overall Length |
|---|---|---|
| I40 to I42 | Ne Douglas St | 81ft |
| L22 to L23 | SE Fir St | 146ft |
| B69 To B70 | Arcadia School Sidewalk | 114ft |
| B39 to B37 | Skyline Hillside Slope | 174ft |
| D9 to D4 | Business 20 | 232ft |
| F17 to F27 | NW 6th St | 184ft |
| C1 TO C18 | Lincoln Way | 32ft |
| M13 to M18 | East Slope Rd | 194ft (22ft unseen) |

Section 6

# 6.0 Rehabilitation Methods

## 6.1 Introduction

This section describes the suitability of various repair methods for sanitary sewer manholes and pipe. Generally speaking, pipe can be lined, patched in place or completely replaced. Each of these can be accomplished through a variety of methods which will be discussed below. Deficient manholes can be reinforced, lined or replaced.

## 6.2 Lining

### 6.2.1 CIPP

Cured-in-place-pipe (CIPP) is a process of manufacturing a replacement pipe within the existing pipe. An impermeable “bag” that contains a sewn tube of non-woven felt fabric is impregnated with a resin that can be activated by hot water or steam. This “bag” is inserted through a manhole and inverted within the host pipe to be repaired. Once inside the pipe, the bag is filled with water or air pressure to expand the liner within the host pipe much like blowing up a balloon. The new pipe material conforms to the outside of the existing pipe and creates a new one-piece pipe liner continuous to the next manhole. The resins are activated by hot water or steam inside the bag which causes the fabric and resin to cure and create the new pipe. A robotic cutting tool is used to open the lateral connections again.

Some of the major benefits of CIPP are:

- All surface excavations and surface restorations are eliminated
- The process is fast and costs are significantly reduced
- All existing joints are sealed
- The new pipe forms limited bonds to the existing pipe which helps prevent I/I migration to the manhole.

Manufacturers claim that CIPP pipe longevity testing shows a lifespan in excess of 50 years.

CIPP cannot repair all problems in a broken host pipe. Large voids or holes in the pipe must be patched prior to the liner installation. If the host pipe contains major grade changes or collapsed sections the liner will either conform to them or not form correctly. CIPP liners are best suited to repairing minor structural problems, leaking joints, minor misalignments, or root penetrations.

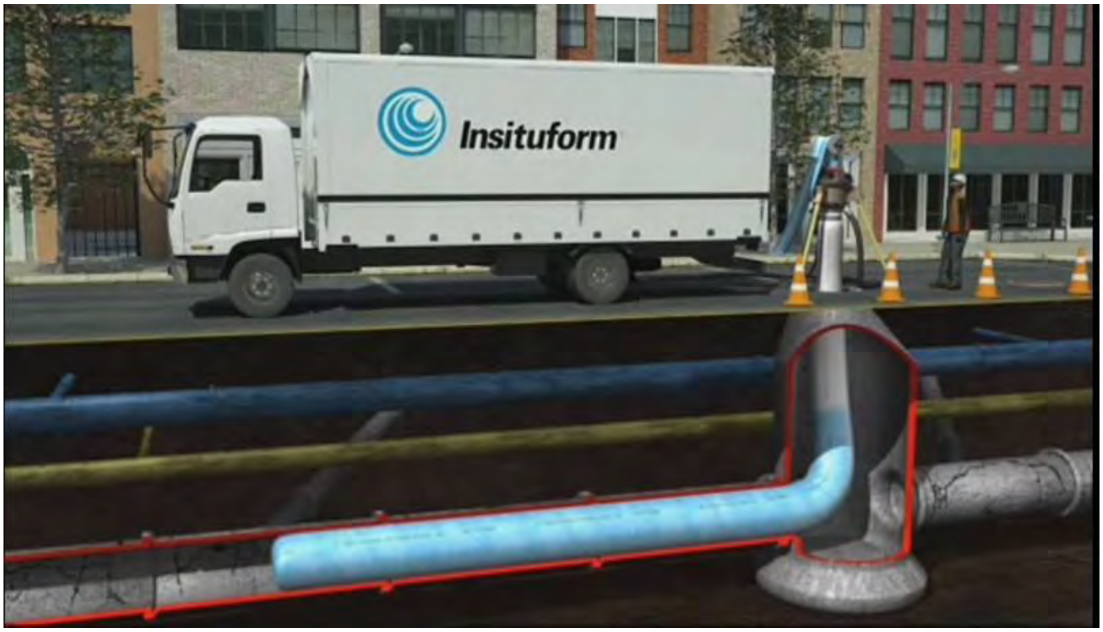



**Figure 6.2.1 CIPP Liner Installation**

### 6.2.2 Slipliner

Sliplining is a process where an entirely new pipe is pulled into an existing pipe. Insertion and receiving pits are dug at both ends of the pipe and a smaller diameter pipe is inserted into the insertion pit which is then pulled through the old pipe into the receiving pit. HDPE pipe is typically used and is either grout sealed at both ends or the grout is pumped in to fill the annular space between both pipes.

CIPP has mostly replaced sliplining for sewer pipe. Major disadvantages of sliplining are:

- A diameter reduction in the new pipe (partially offset by reduced friction)
- The joints on the endpoints can fail and allow the infiltration back in.

Sliplining requires excavations to remake a lateral connection which creates another drawback. As there is little cost difference between the two lining methods, CIPP will be recommended when lining is the most cost effective repair method.

### 6.2.3 Fold & Form

Fold & Form pipe is a PVC pipe which takes advantage of the thermoplastic memory properties inherent in PVC. A folded pipe is inserted into a manhole and pulled through the existing pipe. Both ends of the pipe are plugged and expanded with steam and pressure. Finally the pipe is cooled and maintains its cylindrical shape, resulting in a new jointless PVC liner. Laterals are reconnected in the same manner as a CIPP liner.

Fold & Form pipe requires a slightly thicker wall to have equivalent strength to CIPP liners. As costs are similar it can be considered an alternative to CIPP if local availability or economics favor it.

## 6.3 Patching

### 6.3.1 CIPP

A common tool available for spot repairs in otherwise sound pipe are CIPP pipe patches. They are shorter versions of the liners and are inserted with robotic equipment. These patches are made of the same material and can be inserted and cured in a few hours restoring the integrity of the pipe. Sections can be either field cut to length, or precut sections can be joined together to form a longer patch.

An advantage of using spot repair CIPP patches is that they can be underinflated around pipe voids to reinforce a pipe prior to a full liner being inserted. This can prevent "ballooning" pockets of the main liner when it is pressurized to conform to the pipe wall.

Figure 6.3.1.2 CIPP Patch

### 6.3.2 Open Trench Spot Repairs

The dig and replace method of pipe repair is a good option where surface improvements are minimal or the pipe grade rules out the use of trenchless repair methods. Televising data should be consulted first to determine the nature of the repair. This method is commonly used for emergency repairs where a small section of pipe is exposed and patched with PVC pipe or when new laterals are added into the mainline.

## 6.4 Pipe Replacement

### 6.4.1 Open Trench

Open trench construction is the most basic method of constructing new pipe section or replacing old ones. A trench is excavated to an adequate depth to maintain sufficient gravity drainage slope and allow room to properly bed and access the pipe. Typically, the trench is at least 18 inches wider than the pipe diameter at the base and gradually widens at the top as the overall depth increases. The width of the top of the trench can vary greatly due to soil conditions.

**Figure 6.4.1 Open Trench Pipe Construction**

The advantages of open trench construction include:

- Utilizes common installation techniques available to local contractors
- The ability to adjust and level the pipe grade
- Greater flexibility in adjusting for unforeseen subsurface conditions.

Disadvantages of open trench construction include:

- Expensive surface restoration required, especially in roadways
- Open trench shoring required when excavations are deeper than 5 feet or if soil is unstable
- Dewatering equipment is often needed where groundwater is high
- High restoration impact on public and private properties.

Open trench construction is often most cost effective in new construction where preservation of existing facilities is less important. It is also cost effective in rehabilitation for spot repairs or where the existing pipe exhibits grade problems from settling. Open trench construction allows the use of any of the available pipe materials, though the modern material of choice is PVC sewer pipe (3034).

### 6.4.2 Boring

Boring, or directional drilling, is a method where a highly controllable drilling head creates an underground “tunnel” to insert a new pipe underground. An entry hole is bored into the ground and the drilling head is guided to the exit hole. Special electromagnetic tracking tools are utilized to maintain the direction and depth of the bore. The pipe is then attached and pulled back through the bore hole to the entry point. Drilling fluids pumped into the borehole prevent collapse and aid in the drilling process. HDPE pipe is typically used in boring applications.

Advantages of using boring include:

- The ability to insert pipe into high groundwater or under bodies of water
- Minimal impact to the ground surface
- The ability to cut across hills, mountains, and wetland areas

The major disadvantages of boring include:

- Poorer performance in rocky conditions
- Increased cost compared to open trench methods
- Only specialized equipment is capable of boring grades less than 1% for gravity sewer pipe.


**Figure 6.4.2 Pipe Boring**

Directional drilling is typically not used in sewer rehabilitation work unless the conveyance system is re-routed. For new construction, the terrain and existing structures preservation are factors in deciding the cost effectiveness of choosing boring over open trench construction.

### 6.4.3 Pipe Bursting

Pipe bursting is a method of replacing or upsizing an existing pipeline using the old pipe as a conduit. Pipe bursting eliminates trenching and instead requires only small access pits at laterals and the insertion point. Pipe bursting is accomplished by feeding a cable through the pipe and pulling a bursting head back through the host pipe. The bursting head, either hydraulically or through force alone, expands and breaks apart the old pipe compressing it into the old pipe bedding. Simultaneously while bursting the old pipe, new pipe is pulled into the hole behind it. Access pits are dug at laterals to make reconnection with a saddle joint.

The host pipe has to be constructed of a brittle material, such as clay or concrete pipe, to allow the material to shatter and push into the surrounding soil. HDPE and Fusible PVC are two materials used for replacement pipe as a flexible continuous pipe is needed to meet the bending requirements while

**Figure 6.4.3.2 Pipe Bursting Winch**

inserting the pipe. It is common to upsize the existing pipe as much as 25%, however this capability varies greatly based upon soil conditions, depth of the existing pipe, and available equipment.

**Figure 6.4.3.1 Pipe Bursting Head**

In ideal conditions pipe bursting provides a significant cost savings over open trench methods for rehabilitation. Major advantages of pipe bursting are:

- Can be completed in a matter of hours,
- Only creates small surface disturbances at entry points,
- In many situations new pipe can be pulled directly into the existing manhole,
- A larger pipe can be installed for only minor cost increases.

Disadvantages of pipe bursting are:

- Cannot be used where existing pipe has grade problems,
- Pipelines with dense laterals decrease the cost benefit,
- Only useful in brittle host pipes,
- Cannot be used if sensitive utilities or structures are known to be near to sewer pipe
- Can create surface upheaval if too shallow.

Other variations of pipe bursting exist, such as pipe splitting and pipe reaming, that provide capabilities conventional pipe bursting does not. Pipe splitting uses a cutting head to split the existing pipe in two instead of expanding the pipe and allows bursting operations in non-brittle pipe types. Pipe reaming is similar to the boring process in reverse, where a cutting tool is pulled through the pipe and grinds it into pieces while pulling a new pipe behind. Drilling fluid carries the old pipe fragments into a receiving pit for disposal. Both of these methods are unnecessary for the types of problems identified in this report so will not be explored further.

## 6.5 Lateral Repair Methods

### 6.5.1 Grout repairs

Sewer service laterals can be grout repaired within approximately 2 feet of the mainline connection. Grout repairs are non-disruptive to the service and are completed from within the mainline sewer pipe. A robotic joint packer injects grout into voids and cracks. This grout may last for 10 years or longer if properly installed, especially when exposed to consistent moisture. Lateral and joint grouting can be quickly accomplished for several hundred dollars per connection. Based on our experience, grout repairs are often only marginally effective and often do not stand the test of time.

### 6.5.2 Lateral Bursting

Lateral bursting is a smaller scale version of mainline pipe bursting. It is typically provided by plumbing companies to renovate lateral connections for residents. Bursting still requires an excavation at the mainline connection and the associated surface disturbance. This method is not common for municipal projects that are seeking to rehabilitate pipe up to the property line.

### 6.5.3 Lateral Lining

Various types of lateral liners have been in existence for years. They use the same CIPP process for mainlines. One of the major advantages is that the pipe can be restored with little invasive effort all the way into the mainline. Lateral lining systems come in various versions from short "Top Hat" liners which provide a couple feet of liner around the lateral opening to full liners which make a complete connection from the house to the main pipe.

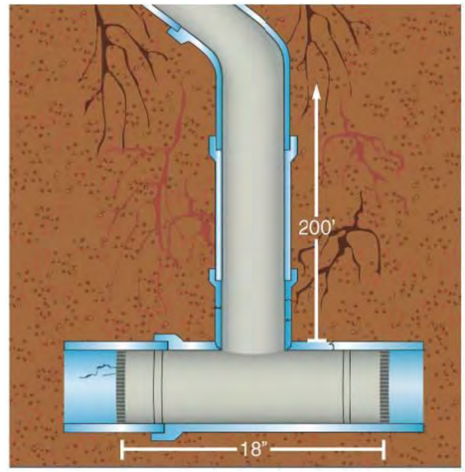


**Figure 6.5.3 Lateral Liner**

Top hat liners have a drawback when used with mainline liners because surface adhesion to cured CIPP pipe is difficult to maintain. A newer system is available where a gasketed tubular connection is made to the mainline and the lateral liner is launched to the lateral cleanout. These liners cost approximately $2500 each and provide a secure connection well beyond the deeper infiltration points. If a cleanout connection does not exist there are options to non-invasively add one. Lateral liners make logical sense when already lining the mainline. However, the high costs of using the liners often make direct placement (dig and replace) of a new lateral more economical.

### 6.5.4 Dig and Replace

Dig and replace is the standard connection method for repairing laterals during open trench replacement or pipe bursting. The lateral is normally replaced up to and including the cleanout at the property line. This approach is generally used when the mainline is being directly replaced.

If utilizing pipe bursting to rehabilitate a sewer mainline, lateral reconnections are typically made using dig and replace methods with access pits at each connection. The best lateral connections to HDPE utilize fusion welded HDPE saddles instead of gasket style saddle. In this report we have assumed that improvements will utilize a fusion welded saddle connected to a new cleanout with either a PVC or HDPE lateral.

## *6.6 Manhole Repair Methods*

Manholes can be rehabilitated in a variety of ways with methods such as coating, lining, grouting and complete replacements.

### 6.6.1 Manhole Sealing

A variety of coatings which can be applied either as spot repairs or a complete vacuum testing sealant are available. Costs can range from $125 to $300 per vertical foot depending upon the process used.

For sealing and repairing manholes which are not exposed to chemical deterioration, a less expensive urethane based sealant can be used. These grouts can be applied as a spray, injection, brushed or mixed to a foam consistency. Urethane type grouts provide the best performance when they are continually exposed to moisture and do not dry out. These grouts can be injected into voids and cracks in the manholes and prevent moisture from coming in. Urethane style grouts have a poor long term performance as a surface coat and would not be recommended for extensive repair work, especially where exposed to hydrogen sulfide deterioration.

**Figure 6.6.1.1 Epoxy Sealed Manhole**

For superior manhole sealing, a fiber reinforced cementitious mortar can be sprayed or troweled onto the manhole surface. The best products provide an extremely strong bond to the existing manhole wall creating a new smooth surface which reinforces the entire structure. They also provide good chemical resistance to the manhole wall. As a product group the cementitious mortars have a higher level of success than urethane systems, but some products perform much better than others and well trained applicators are important. The City should carefully review product data before selecting a contractor.

The most expensive and best methods for manhole sealing are epoxy based coatings. These are ideal for situations where consistently high levels of hydrogen sulfide exposure are present. One cost savings method is to apply a fiber reinforced mortar as a base coat to the manhole for filling of voids and use an epoxy sealant as a top coat. Coating manholes with epoxy can cost nearly as much as a new manhole, causing this option to only be viable in specific situations

**Figure 6.6.1.2 Cementious Mortar Spot Repaired Manhole**

### 6.6.2 Manhole Liners

Fiberglass style liners are available to reinforce and seal existing manholes. Rather than being sprayed or troweled on like sealers, these liners are structural materials that are placed into the manhole and forming a new "manhole within the manhole". A variety of processes are used to accomplish this, some are premade while others are formed with a CIPP style process. It is approximately $300 per vertical foot to line a standard 48" manhole. This is only slightly less than constructing a new manhole under normal circumstances.

**Figure 6.6.2 Manhole Liner**

### 6.6.3 Manhole Replacement

New concrete or HDPE (high density polyethylene) manholes can be installed where an existing manhole has failed. The cost to replace a manhole can range from $4000-$5000 and may be the best choice when doing open trench construction for a long pipe section.

Section
7

# 7.0 Improvement Projects

## 7.1 Introduction

This section describes in detail grouped repair projects chosen from the combined results of smoke testing, flow mapping and televising.

Improvement projects have been categorized by recommended repair type and geographical proximity. Repair types have been selected based upon pipe conditions, surface condition, I/I levels and overall cost effectiveness. All deficient pipelines and manholes can be suitably replaced using the open trench method, but this method was not recommended unless pipe grade, surface conditions, or pipe failures have made it necessary to forego lower cost trenchless options. A few of the open trench projects were incompletely inspected, however the inspected portion of the pipe was often judged to be in such poor condition that further inspection would be unlikely to change the recommendation.

GIS mapping with exact manhole and pipeline locations is not available for Toledo. In order to assist with finding repair locations, each project has an aerial map with an approximate location of the line drawn on it. A table showing manhole numbers was created as part of the Smoke Testing Survey and added to the City's mapping is also included in each estimate. The existing manhole and sewer network mapping maintained by City is generally accurate and if inconsistencies were found, during the flow mapping and smoke testing surveys, we revised the mapping to show the correct flow directions and manhole connections.

## 7.2 Discussion of Cost Estimates

Cost estimates for the projects in this section include several items. Once the preferred repair method was chosen, the associated improvements and local area conditions were considered when developing cost estimates for the repairs. The restoration of any structures or landscapes, if found to be significant, were also included in the estimates.

Mobilization and temporary facilities costs are based upon a percentage of the cost of the estimated construction work. Mobilization includes the cost to move and rent equipment as well as many one-time costs associated with starting and ending a construction job. Temporary facilities include items such as fencing, traffic control, restrooms, markers and erosion control objects. Adjustments of these prices have been made when items such as specialized equipment are needed for a small job or the project includes repairs over a wider geographic area.

Project estimates include three cost totals. The construction cost total is the estimate of all the individual tasks required to complete the project. The subtotal is the construction cost total added to a contingency percentage factor based upon the construction costs. The final cost is the total project cost, which includes engineering and administrative percentage factors based upon the subtotal cost.

Contingency costs are intended to account for unknowns. At this stage of the process the improvement projects have not included subsurface geotechnical surveys, sewer laterals have not been thoroughly checked, easements status not been verified and the required design surveys are not complete. As the projects continue through the design process and approach the construction phase, the number of unknowns will diminish and allow the contingency factor to decrease. Contingency costs have been set to 25% of the construction cost estimate for this study.

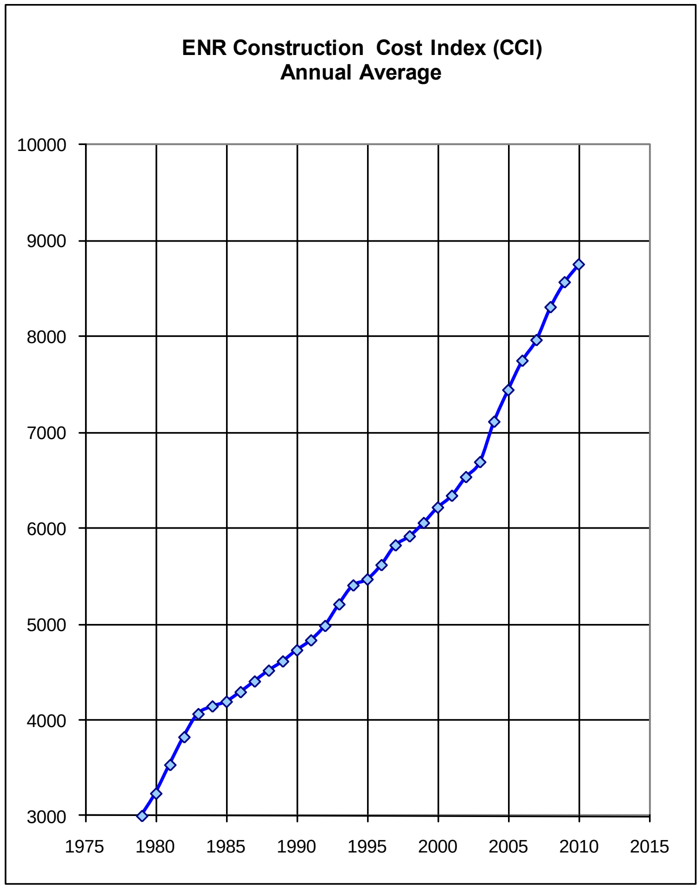



**Figure 7.2**

Engineering fees are estimated as a percentage of the subtotal cost, typically around 20%. Presumably, events or unknowns accounted for by contingencies will likewise incur additional engineering and administrative charges. The engineering time required will vary based upon many factors but generally more complex projects with higher requirements are more costly than others.

Administrative costs consist of a small portion of the overall project price. They include items such as legal fees, city staff costs, and the cost of obtaining the required permits, internal planning and any miscellaneous non-construction related work. Administrative costs in this report have been estimated at 3% of the subtotal cost.

Cost estimates for the construction portion of each of the projects have been based upon pricing for similar recent projects and material estimates from suppliers. These estimates utilize broader categories with higher costs than would be typical of a bid item list. Further engineering of each project will refine the estimates.

Over time, prices typically increase as inflation reduces the value of money. In order to allow budget planning in the future for the projects prepared in this report, the projects can be compared to the Engineering News Record (ENR) Construction Cost Index (CCI).

The ENR CCI provides an index numbering system that allows conversion of project costs across time periods. Construction costs of projects are determined monthly and assigned a number relative to an absolute baseline year cost.

The ENR Construction Cost Index uses an established value of 100 for the year 1913. The index value for November 2010 used in this report is 8951. For instance, if a project cost $10,000 to construct in 1913, the cost to construct it today would be $895,100 based upon growth in the ENR CCI. A graph is presented in Figure 7.2 which shows the ENR CCI recent trends.

Over the last 10 years the ENR index has grown approximately 3.5% per year. If that trend continues, a $100,000 project in this report will cost approximately $111,000 in three years and $141,000 in ten years to complete.

## *7.3 Project List*

### 7.3.1 Pipe Patching Project A

A single project is proposed to cost effectively patch pipes throughout the City. Many of these locations are structurally intact pipes with a single break or a poor joint. A patch should seal the infiltration and may allow the pipe to remain in service for many years.

A mixture of non-invasive CIPP pipe patches, CIPP Lateral liners, and invasive dig and repair sections are included within this project. Areas where a short pipe belly or large offset exists are recommended for excavated patches while those pipes with holes and bad leaks are recommended for CIPP repair methods.

None of these pipes are in excellent condition and we would expect that they should be re-inspected in 10 years to observe if any new deficiencies have formed. Ultimately only the lined laterals will provide service for a substantial length of time and it is likely some of these pipe segments will be replaced over the next two decades.

**TABLE 7.2.1.1 – PATCHING PROJECT, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| C5 to C6 | Two 15 foot belly repairs, open trench PVC |
| C21 to C18 | CIPP Pipe Patch |
| B16 to B12 | CIPP Pipe Patch, Lateral CIPP Patch |
| O7 to O6 | Protruding lateral cut and re-grout |
| F41 to F38 | CIPP Lateral Patch, 10 foot open trench PVC repair belly into manhole F38 |
| I23 to I84 | 2 CIPP Pipe Patches, Cut and spray 3 root joints and grout |
| I19 to I18 | Lateral CIPP Patch |
| K16 to K18 | 5 foot offset pipe, open trench PVC repair |
| F34 to F9 | 10 Foot open trench PVC belly repair, Lateral CIPP Patch |
| O12 to O7 | Cut and Spray 2 root joints and grout, Protruding lateral cut and grout |

**TABLE 7.2.1.2 – PATCHING PROJECT, COST ESTIMATE**

| Patching Project #A | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $4,000.00 | $4,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $3,000.00 | $3,000.00 |
| 3 | CIPP Lateral Liner | ea | 4 | $2,500.00 | $10,000.00 |
| 4 | Cut and Grout | ea | 2 | $500.00 | $1,000.00 |
| 5 | CIPP Pipe Patch | ea | 4 | $2,500.00 | $10,000.00 |
| 6 | Cut roots and grout joint | ea | 5 | $350.00 | $1,750.00 |
| 7 | Asphalt Trench Patch | sq yds | 20 | $60.00 | $1,200.00 |
| 8 | Open Trench Patch 8" PVC | lf | 55 | $80.00 | $4,400.00 |
| 9 | Surface Restoration | ls | 1 | $3,000.00 | $3,000.00 |
| | | Construction Total | | | $38,350.00 |
| | | Contingency (25%) | | | $10,000.00 |
| | | Subtotal | | | $48,350.00 |
| | | Engineering (20%) | | | $9,700.00 |
| | | Administrative Costs (3%) | | | $1,500.00 |
| | | Total Project Costs | | | $59,550.00 |


**MAP 7.3.1.1 PATCHING PROJECT A (NORTH AREA)**


**MAP 7.3.1.2 PATCHING PROJECT A (SOUTH AREA)**

### 7.3.2 North Nye Street Project B

Under the northern gravel portion of North Nye Street, at the base of the hill coming down from Skyline Drive, is a long pipe segment containing several holes with high infiltration. Our flow mapping inspection resulted in the measurement of a considerable amount of infiltration isolated to this pipe segment. In addition, several of the laterals connecting to the pipe exhibited high clear flows during television inspection. The combination of the high infiltration and broken pipe suggests that this pipe segment ought to have the highest priority of the non-critical segments to repair.

The pipe is constructed of concrete and includes an ABS patch; likely a repair to a previous leak or hole. It was observed that the pipe is buried over 10 feet deep. Because of the type of residential neighborhood with widely spaced homes, some of the lateral connections are very long.

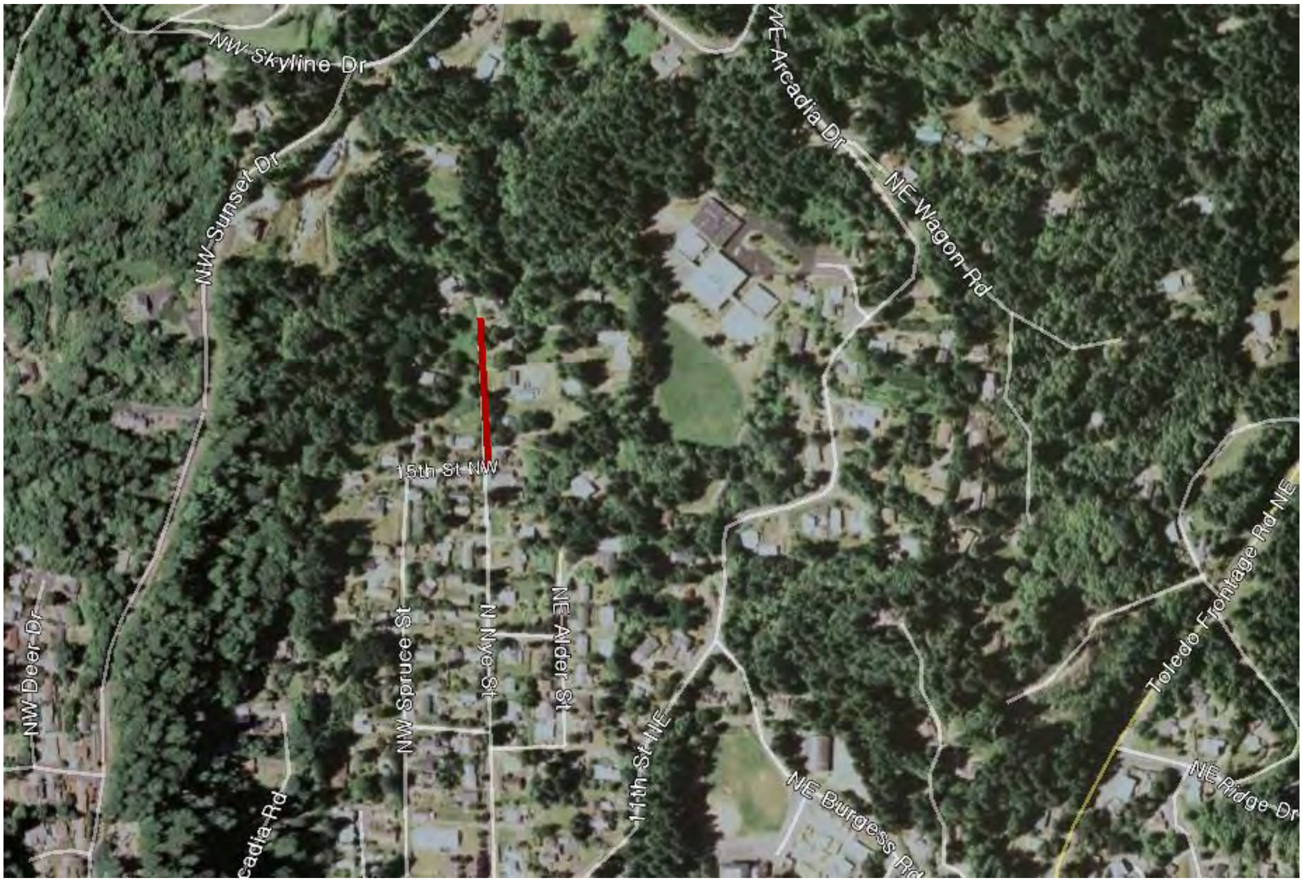
The recommendation, for this project, is to dig and replace this pipe due to its placement in aggregate and to allow investigation of the significant lateral leaks. Laterals should be replaced to the property lines. It is further recommended to televise the laterals, including the portion on private property, to further investigate where high infiltration is originating. The City may find it needs to require property owners to repair or replace their laterals.

**TABLE 7.2.2.1 – NORTH NYE STREET, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| B39 to B31 | Pipe Replacement |

**TABLE 7.2.2.2 – NORTH NYE STREET, COST ESTIMATE**

| N Nye St Replacement Project #B | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $11,000.00 | $11,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $8,000.00 | $8,000.00 |
| 3 | 8" PVC Pipe (entire pipe >10’ deep) | lf | 464 | $95.00 | $44,080.00 |
| 4 | New Manhole | ea | 2 | $9,000.00 | $9,000.00 |
| 5 | Lateral Connections | ea | 9 | $3,000.00 | $27,000.00 |
| 6 | Lateral Televising | ea | 9 | $150.00 | $1,350.00 |
| 7 | Aggregate Trench Patch | tons | 592 | $25.00 | $14,800.00 |
| | | Construction Total | | | $115,230.00 |
| | | Contingency (25%) | | | $29,000.00 |
| | | Subtotal | | | $144,230.00 |
| | | Engineering (20%) | | | $28,900.00 |
| | | Administrative Costs (3%) | | | $4,400.00 |
| | | Total Project Costs | | | $177,530.00 |




MAP 7.3.2 N NYE ST REPLACEMENT PROJECT B

### 7.3.3 Northeast 12th Street Project C

Three short pipe segments under Northeast 12th Street have been combined into a single repair project. A combination of pipe bellies, cracks, large root penetrations and many leaking joints are affecting this area. Several of the laterals are heavily leaking. Problems were noted in both smoketesting and flowmapping with verification seen during television inspection.

It is recommended to dig and replace the pipes to grade. Some locations of the pipe require asphalt patch where the pipe is located in the roadway. It is also anticipated that one of the manholes will need to be replaced to re-grade the pipe segments, especially from manhole B16 to B18.

Alignment of the sewer lines here appears to follow the grassy shoulder beside the road, however estimates assume a complete asphalt trench patch.

**TABLE 7.2.3.1 – NE 12TH STREET, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| B20 to B18 | Pipe Replacement |
| B20 to B22 | Pipe Replacement |
| B16 to B18 | Pipe Replacement |

**TABLE 7.2.3.2 NE 12TH STREET, COST ESTIMATE**

| NE 12th St Project #C | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $9,000.00 | $9,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $6,000.00 | $6,000.00 |
| 3 | 8" PVC Pipe | lf | 386 | $85.00 | $32,810.00 |
| 4 | New Manhole | ea | 2 | $9,000.00 | $4,500.00 |
| 5 | Lateral Connections | ea | 7 | $3,000.00 | $21,000.00 |
| 6 | Asphalt Trench Patch | sq yds | 257 | $60.00 | $15,420.00 |
| | | Construction Total | | | $88,730.00 |
| | | Contingency (25%) | | | $23,000.00 |
| | | Subtotal | | | $111,730.00 |
| | | Engineering (20%) | | | $22,400.00 |
| | | Administrative Costs (3%) | | | $3,400.00 |
| | | Total Project Costs | | | $137,530.00 |


**MAP 7.3.3 NE 12TH ST PROJECT C**

### 7.3.4 Southeast 10th Street Project D

The pipe segment traveling down the slope of Southeast 10th Street toward the Olalla Creek bridge showed considerable signs of inflow during smoketesting. Extremely heavy roots and deposit buildup were found in subsequent televising. The pipe itself is in very poor condition and urgent replacement is recommended.

Pipe bursting is recommended to avoid replacing the edge of the pavement and curb. There are few lateral connections in this pipe segment but they each should be replaced with PVC to the property line and connected to a fusion welded HDPE saddle.

During flow mapping and smoketesting there was some confusion related to unexpected manholes on this hillside. It is recommended that the City update their internal mapping to better show the pipe and manhole connections along this street.

**TABLE 7.2.4.1 – SE 10TH STREET, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| N3 to N4 | Pipe Bursting |

**TABLE 7.2.4.2 – SE 10TH STREET, COST ESTIMATE**

| SE 10th St Project #D | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $4,000.00 | $4,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $3,000.00 | $3,000.00 |
| 3 | 8" HDPE Pipe bursting | lf | 292 | $45.00 | $13,140.00 |
| 4 | New Manhole | ea | 1 | $4,500.00 | $4,500.00 |
| 5 | Lateral Connections | ea | 4 | $2,500.00 | $10,000.00 |
| 6 | Surface Restoration | ea | 1 | $3,500.00 | $3,500.00 |
| | | Construction Total | | | $38,140.00 |
| | | Contingency (25%) | | | $10,000.00 |
| | | Subtotal | | | $48,140.00 |
| | | Engineering (20%) | | | $9,700.00 |
| | | Administrative Costs (3%) | | | $1,500.00 |
| | | Total Project Costs | | | $59,340.00 |


**MAP 7.3.4 SE 10TH ST PROJECT D**

### 7.3.5 East Graham Street Project E

Along the steep slope where East Graham Street intersects Main Street, several pipe cracks and root penetrations were discovered. Initially, the pipe was found to contain high infiltration from the Flow Mapping Survey. During televising it was observed that the 10-inch concrete pipe is in serviceable condition at the upper portion and begins to have root joint failure for the lower two-thirds of the pipe.

It was not possible to televise the entire pipe due to a protruding lateral. This lateral should be cut and, once complete, the recommendation is to line the pipe with a CIPP liner.

**TABLE 7.2.5.1 – EAST GRAHAM STREET, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| I34 to I33 | CIPP Liner, Verify remainder of pipe before construction |

**TABLE 7.2.5.2 – EAST GRAHAM STREET, COST ESTIMATE**

| E Graham St Project #E | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $4,000.00 | $4,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $3,000.00 | $3,000.00 |
| 3 | 10" CIPP Liner | lf | 375 | $45.00 | $16,875.00 |
| 4 | CIPP Lateral Liner | ea | 5 | $2,500.00 | $12,500.00 |
| | | Construction Total | | | $36,375.00 |
| | | Contingency (25%) | | | $10,000.00 |
| | | Subtotal | | | $46,375.00 |
| | | Engineering (20%) | | | $9,300.00 |
| | | Administrative Costs (3%) | | | $1,400.00 |
| | | Total Project Costs | | | $57,075.00 |


**MAP 7.3.5. E GRAHAM ST PROJECT E**

### 7.3.6 Northwest 6th Street Project F

6th street has a collapsing pipe at the dead-end intersecting Beech Street. Complete televising of the entire pipe section was not possible due to extreme root intrusion blocking access for the camera equipment. Because the remaining structure of the pipe is unknown, it is recommended to proceed with an open trench replacement in preference to trenchless repairs. Lateral connections are unknown as well and have been assumed based upon nearby residences.

**TABLE 7.2.6.1 – NW 6TH STREET, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| F26 to F23 | Pipe Replacement, root removal before construction and reinspection for design. |

**TABLE 7.2.6.2 – NW 6TH STREET, COST ESTIMATE ALTERNATIVE 1**

| NW 6th St Project, Alternative F1, Open Trench Replacement | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $7,000.00 | $7,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $5,500.00 | $5,500.00 |
| 3 | 8" PVC Pipe | lf | 307 | $85.00 | $26,095.00 |
| 4 | Lateral Connections (assumed) | ea | 4 | $3,000.00 | $12,000.00 |
| 5 | New Manhole | ea | 1 | $4,500.00 | $4,500.00 |
| 6 | Asphalt Trench Patch | sq yds | 200 | $60.00 | $12,000.00 |
| 7 | Landscape Restoration | ls | 1 | $2,000.00 | $2,000.00 |
| | | Construction Total | | | $69,095.00 |
| | | Contingency (25%) | | | $18,000.00 |
| | | Subtotal | | | $87,095.00 |
| | | Engineering (20%) | | | $17,500.00 |
| | | Administrative Costs (3%) | | | $2,700.00 |
| | | Total Project Costs | | | $107,295.00 |

A second cost estimate has been developed to include an alternative pipe bursting repair. This second estimate has been provided as a potential lower cost repair if further investigation is completed. This estimate includes further cleaning and inspection of the pipe and makes the assumption that the pipe segment will be found in adequate condition to burst.

It is possible televising and root cutting measures will conclude the pipe cannot be repaired using non-invasive methods and Alternative F1 must be used anyway.

**TABLE 7.2.6.3 – NW 6TH STREET, COST ESTIMATE ALTERNATIVE 2**

| NW 6th St Project, Alternative F2, Pipe Bursting | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $4,000.00 | $4,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $3,000.00 | $3,000.00 |
| 3 | 8" HDPE Pipe Bursting | lf | 307 | $45.00 | $13,815.00 |
| 4 | Lateral Connections (assumed) | ea | 4 | $3,000.00 | $12,000.00 |
| 5 | New Manhole | ea | 1 | $4,500.00 | $4,500.00 |
| 6 | Root Cutting & Re-Televising | lf | 292 | $2.00 | $600.00 |
| 7 | Surface Restoration | ls | 1 | $3,000.00 | $3,000.00 |
| | | Construction Total | | | $40,915.00 |
| | | Contingency (25%) | | | $11,000.00 |
| | | Subtotal | | | $51,915.00 |
| | | Engineering (20%) | | | $10,400.00 |
| | | Administrative Costs (3%) | | | $1,600.00 |
| | | Total Project Costs | | | $63,915.00 |


**MAP 7.3.6 NE 6TH ST PROJECT F**

### 7.3.7 Business 20 Replacement Project G

Heavily bellied pipe is buried under Business 20 near the police station. This pipe was suspected of heavy flows during flow mapping. Television inspection was unsuccessful due to very poor pipe grade forcing the camera underwater through most of the survey. The portions that were visible contained heavy leaks at every joint. The current pipe is 8-inch concrete and observed flow lines indicate a full pipe is often experienced in this section.

Significant settlement is occurring in the pipe along its current alignment, likely due to its placement near a tidal lowland area. There is also concern that the sanitary sewer mapping shows the pipe could be located underneath an existing building. We did consider moving the alignment north and routing the pipeline under Business 20 until its intersection with "A" Street. The "A" street intersection is on a rising slope resulting in the realignment having a depth of approximately 20 feet at the terminating manhole.

Feedback received from long time Public Works Department employees suggest that the existing alignment is located between existing buildings, not beneath them. Our recommendation is to replace the existing pipeline using the current alignment which will reduce traffic disruption, require less asphalt patching, and not require deep trenching equipment. We do anticipate that some foundation stabilization and dewatering equipment will be necessary at this site.

This project includes the replacement of 4 pipe segments and installation of 4 new manholes.

**TABLE 7.2.7.1 – BUSINESS 20 REPLACEMENT, PIPES SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| D1 to F8 | Realign, upsize to 10-inch, eliminate belly |
| D1 to D2 | Realign, upsize to 10-inch, eliminate belly |
| D2 to D3 | Realign, upsize to 10-inch, eliminate belly |
| D3 to D4 | Realign, upsize to 10-inch, eliminate belly |

**TABLE 7.2.7.2 – BUSINESS 20 REPLACEMENT, COST ESTIMATE**

| Business 20 Project #G | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $12,000.00 | $12,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $9,000.00 | $9,000.00 |
| 3 | 10" PVC Pipe | lf | 602 | $95.00 | $57,190.00 |
| 4 | Asphalt Trench Patch | sq yds | 200 | $60.00 | $12,000.00 |
| 5 | Foundation Stabilization | cu yds | 100 | $36.00 | $3,600.00 |
| 6 | Dewatering | ea | 1 | $5,000.00 | $5,000.00 |
| 7 | New Manhole | ea | 4 | $4,500.00 | $18,000.00 |
| 8 | Landscape Restoration | ea | 1 | $6,000.00 | $6,000.00 |
| | | Construction Total | | | $122,790.00 |
| | | Contingency (25%) | | | $31,000.00 |
| | | Subtotal | | | $153,790.00 |
| | | Engineering (20%) | | | $30,800.00 |
| | | Administrative Costs (3%) | | | $4,700.00 |
| | | Total Project Costs | | | $189,290.00 |


**MAP 7.3.7 BUSINESS 20 REPLACEMENT PROJECT G**

### 7.3.8 Southeast 5th Street Project

5th Street sewer pipe is full of roots and the pipe itself appears to be worn past its useful life. A large hole exists near one end and large deposits have blocked part of the pipe. Most of the pipe was able to be observed in spite of the obstruction. The 8-inch concrete pipe is recommended to be repaired with a CIPP liner.

Many of the laterals were observed to be likely I/I contributors. It is recommended that the laterals be rehabilitated or replaced following the main line CIPP rehabilitation. This may be accomplished through the use of a lateral liner system or a direct installation of a new “cut-in” tee and lateral piping. The most cost effective approach should be identified during final design.

**TABLE 7.2.8.1 – SE 5th STREET, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| K29 to K28 | CIPP Liner, Lateral repairs. Recommend eliminate blockage and inspect remainder of pipe |

**TABLE 7.2.8.2 – SE 5TH STREET, COST ESTIMATE**

| SE 5th St Project | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $3,500.00 | $3,500.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $2,000.00 | $2,000.00 |
| 3 | 8" CIPP Liner | lf | 335 | $40.00 | $13,400.00 |
| 4 | CIPP Lateral Liner | ea | 4 | $2,500.00 | $10,000.00 |
| | | Construction Total | | | $28,900.00 |
| | | Contingency (25%) | | | $8,000.00 |
| | | Subtotal | | | $36,900.00 |
| | | Engineering (20%) | | | $7,400.00 |
| | | Administrative Costs (3%) | | | $1,200.00 |
| | | Total Project Costs | | | $45,500.00 |


**MAP 7.3.8 SE 5TH ST PROJECT H**

### 7.3.9 Southeast Alder Street Project I

Two small pipe segments on Alder Street are recommended for lining. The pipes themselves are in rough condition and a large hole along with root intrusion is evident. As lateral problems were not observed in any of the surveys, liner connections are rehabilitated with grouting methods.

Obstacles were noted in the pipe during television inspection. Before the liner is installed it should be properly cleaned and re-televised to ensure the pipe is clear and no blockages will impede the installation. Estimates also include installing a pipe patch prior to installing the liner over the large hole. The patch may not be necessary and a liner installer should be consulted prior to construction.

**TABLE 7.2.9.1 – SE ALDER STREET, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| I29 to I28 | CIPP Liner, Possible CIPP Patch at hole before Lining |
| I28 to I27 | CIPP Liner |

**TABLE 7.2.9.2 – SE ALDER ST, COST ESTIMATE**

| SE Alder St Project #I | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $3,500.00 | $3,500.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $2,000.00 | $2,000.00 |
| 3 | 8" CIPP Liner | lf | 274 | $40.00 | $10,960.00 |
| 4 | CIPP Patch | ea | 1 | $2,500.00 | $2,500.00 |
| 5 | Lateral Grout connections | ea | 9 | $300.00 | $2,700.00 |
| | | Construction Total | | | $21,660.00 |
| | | Contingency (25%) | | | $6,000.00 |
| | | Subtotal | | | $27,660.00 |
| | | Engineering (20%) | | | $5,600.00 |
| | | Administrative Costs (3%) | | | $900.00 |
| | | Total Project Costs | | | $34,160.00 |


**MAP 7.3.9 SE ALDER ST PROJECT I**

### 7.3.10 Butler Bridge Slope Project J

Slopes above Butler Bridge Road drain a small portion of the City with a pipeline portion known as the "Robert's" line. During smoketesting significant quantities of smoke were returned in the heavily forested area. Due to bolted manholes, this area was not able to be properly surveyed during flow mapping. During television inspection the pipe was so heavily rooted that the camera could not travel more than one segment without becoming stuck.

The pipeline is a known maintenance problem with a scheduled flushing interval. Because of the relative condition of the pipes, and the unknown condition combined with the smoketesting results, the recommendation is to replace all the piping and manholes on the hillside. Open trench replacement is used due to uncertainty for pipe bursting conditions.

**TABLE 7.2.10.1 – BUTLER BRIDGE, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| K16 to K15 | Pipe Replacement |
| K15 to K14 | Pipe Replacement |
| K14 to K13 | Pipe Replacement |
| K13 to K12 | Pipe Replacement |
| K12 to K11 | Pipe Replacement |
| K11 to K3 | Pipe Replacement |

**TABLE 7.2.10.2 – BUTLER BRIDGE, COST ESTIMATE #1**

| Butler Bridge Slope Project, Alternative J1 - Open Trench Replacement | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $14,000.00 | $14,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $11,000.00 | $11,000.00 |
| 3 | 8" PVC Pipe | lf | 960 | $85.00 | $81,600.00 |
| 4 | New Manhole | ea | 5 | $4,500.00 | $22,500.00 |
| 5 | Landscape Restoration | ls | 1 | $10,000.00 | $10,000.00 |
| | | Construction Total | | | $139,100.00 |
| | | Contingency (25%) | | | $35,000.00 |
| | | Subtotal | | | $174,100.00 |
| | | Engineering (20%) | | | $34,900.00 |
| | | Administrative Costs (3%) | | | $5,300.00 |
| | | Total Project Costs | | | $214,300.00 |

An alternative to open trench replacement is to quickly pipe burst each of the pipe segments. In order for this to be possible, the heavy root intrusion must be cut and the pipe grade and condition re-analyzed. Deficient manhole replacement and major disruption to the landscaping would continue to result. If the pipe condition is suitable for bursting, cost savings would be realized through the quicker installation speed of fused HDPE pipe. It is emphasized that further analysis may not conclude this is a suitable pipe bursting or lining project in which case open trench replacement would be required.

**TABLE 7.2.10.3 – BUTLER BRIDGE, COST ESTIMATE #2**

| Butler Bridge Slope Project, Alternative J2 - Pipe Bursting | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $9,000.00 | $9,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $7,000.00 | $7,000.00 |
| 3 | 8" HDPE Pipe | lf | 960 | $45.00 | $43,200.00 |
| 4 | New Manhole | ea | 5 | $4,500.00 | $22,500.00 |
| 5 | Root Cutting and Re-Televising | lf | 960 | $2.00 | $1,920.00 |
| 6 | Landscape Restoration | ls | 1 | $10,000.00 | $10,000.00 |
| | | Construction Total | | | $93,620.00 |
| | | Contingency (25%) | | | $24,000.00 |
| | | Subtotal | | | $117,620.00 |
| | | Engineering (20%) | | | $23,600.00 |
| | | Administrative Costs (3%) | | | $3,600.00 |
| | | Total Project Costs | | | $144,820.00 |


**MAP 7.3.10 BUTLER BRIDGE SLOPE PROJECT J**

### 7.3.11 North Main Street Project K

A small pipe segment just north of Business 20 on Main Street is experiencing broken and leaking joints. Because it is short and in reasonable condition this pipe segment is recommended for lining. Both laterals are also leaking and suggested to have lateral liners installed.

A second pipe on the opposite side of the hill is in considerably better condition. However, this pipe contains many leaking joints and should be lined as well. Both pipe segments have been combined into this project.

**TABLE 7.2.11.1 - NORTH MAIN, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| I81 to I78 | CIPP Pipe Liner |
| F20 to F18 | CIPP Pipe Liner |

**TABLE 7.2.11.2 - NORTH MAIN, COST ESTIMATE**

| N Main St Project #K | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $3,000.00 | $3,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $2,000.00 | $2,000.00 |
| 3 | 8" CIPP Liner | lf | 258 | $40.00 | $10,320.00 |
| 4 | CIPP Lateral Liners | ea | 2 | $2,500.00 | $5,000.00 |
| | | Construction Total | | | $20,320.00 |
| | | Contingency (25%) | | | $6,000.00 |
| | | Subtotal | | | $26,320.00 |
| | | Engineering (20%) | | | $5,300.00 |
| | | Administrative Costs (3%) | | | $800.00 |
| | | Total Project Costs | | | $32,420.00 |


**MAP 7.3.11 NORTH MAIN ST PROJECT K**

### 7.3.12 Business 20 Bursting Project L

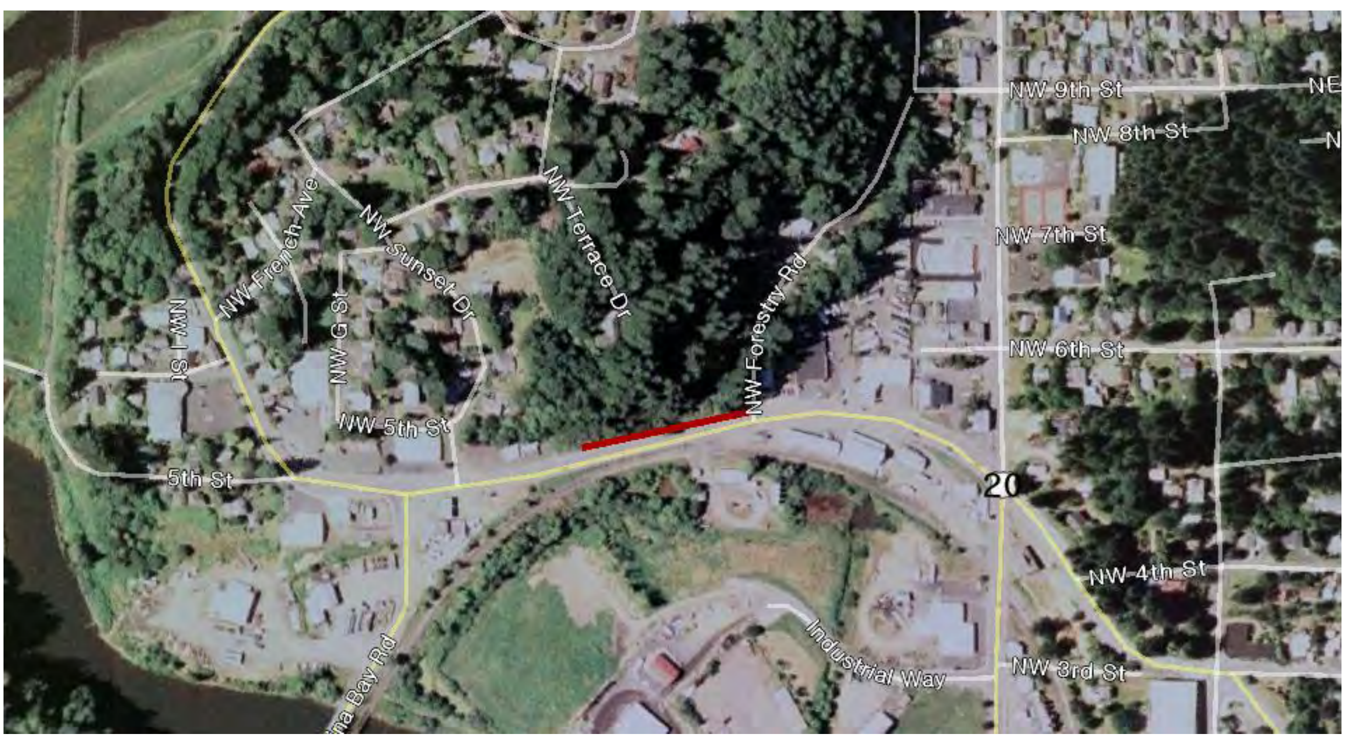
One portion of pipe along Business 20 with many leaks is a good candidate for pipe bursting. The pipe is in reasonable structural condition and no major bellies. High flow lines likely indicate that the pipe capacity is often reached so the recommendation is to increase the size. This project should not be considered urgent but is contributing noticeable I/I to the system.

**TABLE 7.2.12.1 - BUSINESS 20, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| D11 to D9 | Pipe Bursting, upsize to 10-inch |

**TABLE 7.2.12.2 – BUSINESS 20, COST ESTIMATE**

| Business 20 Bursting Project #L | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $4,000.00 | $4,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $3,000.00 | $3,000.00 |
| 3 | 10" HDPE Pipe Bursting | lf | 382 | $55.00 | $21,010.00 |
| 4 | Surface Restoration | ls | 1 | $3,000.00 | $3,000.00 |
| | | Construction Total | | | $31,010.00 |
| | | Contingency (25%) | | | $8,000.00 |
| | | Subtotal | | | $39,010.00 |
| | | Engineering (20%) | | | $7,900.00 |
| | | Administrative Costs (3%) | | | $1,200.00 |
| | | Total Project Costs | | | $48,110.00 |




**MAP 7.3.12 Business 20 Bursting Project L**

### 7.3.13 Alley Repair Project M

A known "bad pipe" is in an alley type area behind a building downtown. This alley aligns north and south parallel to Main Street. Severe smoke testing problems were observed in this immediate area. When televising was performed the survey was obstructed due to large concrete pieces, possibly pieces of pipe, inside. The portion of the pipe that could be observed contains roots and leaking joints.

The City Public Works employees have indicated that this pipe has been bypassed and the laterals it services no longer used. Two cost estimates have been prepared. One in Table 7.2.13.2 assumes that the pipe is not in use and requires plugging to stop I/I flow. The other estimate in Table 7.2.13.3 assumes that the laterals are still required and the pipe needs replacement, including restoration of the parking lot and retaining wall above the pipe.

**TABLE 7.2.13.1 – ALLEY REPAIR, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| I69 to I74 | Pipe Replacement, Further Investigation |

**TABLE 7.2.13.2 – ALLEY REPAIR, PLUG & ABANDON ESTIMATE**

| Alley Repair Project, Alternative #1M, Pipe Abandonment | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $700.00 | $700.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $550.00 | $550.00 |
| 3 | Slurry Plug Pipe | lf | 375 | $15.00 | $5,625.00 |
| | | Construction Total | | | $6,875.00 |
| | | Contingency (25%) | | | $1,800.00 |
| | | Subtotal | | | $8,675.00 |
| | | Engineering (20%) | | | $1,800.00 |
| | | Administrative Costs (3%) | | | $300.00 |
| | | Total Project Costs | | | $10,775.00 |

**TABLE 7.2.13.3 – ALLEY REPAIR, REHABILITATE COST ESTIMATE**

| Alley Repair Project, Alternative #2M, Pipe Replacement | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $6,000.00 | $6,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $4,500.00 | $4,500.00 |
| 3 | 8" PVC Pipe | lf | 275 | $85.00 | $23,375.00 |
| 4 | New Manhole | ea | 1 | $4,500.00 | $4,500.00 |
| 5 | Asphalt Trench Patch | sq ft | 184 | $60.00 | $11,040.00 |
| 6 | Landscape Restoration | ls | 1 | $10,000.00 | $10,000.00 |
| | | Construction Total | | | $59,415.00 |
| | | Contingency (25%) | | | $15,000.00 |
| | | Subtotal | | | $74,415.00 |
| | | Engineering (20%) | | | $14,900.00 |
| | | Administrative Costs (3%) | | | $2,300.00 |
| | | Total Project Costs | | | $91,615.00 |


**MAP 7.3.13 ALLEY REPAIR PROJECT M**

### 7.3.14 Alder Way Project N

City collections staff asked that the pipeline under Alder Way be televised. Though some problems were seen during smoke testing, nothing significant was found to suggest major problems with this pipe.

Television inspection confirmed the suspicions of the collections staff. Many deficiencies were found throughout the piping in the Alder Way neighborhood. The deficiencies include rat holes, lateral holes, joint problems, pulled gaskets and very worn pipe. One portion of the pipe has had a partial CIPP liner installed. This liner is in excellent condition and no problems are seen in this part of the pipe.

The recommendation is for a CIPP liner to be installed in the remained of the pipe segments and the laterals to be lined and repaired.

**TABLE 7.2.14.1 – ALDER WAY, PIPE SEGMENTS REQUIRING REPAIR**

| Pipe Segment Manhole to Manhole | Repair Recommendations |
|---|---|
| Cleanout to O-11 | CIPP Liner, CIPP Lateral Repairs |
| O-11 to O-10(not found) | CIPP Liner, CIPP Lateral Repairs |
| O-10(not found) to O-9 | CIPP Liner, CIPP Lateral Repairs |
| O-9 to O-8(not found) | CIPP Liner, CIPP Lateral Repairs |
| O-8(not found) to O-7 | Partial CIPP liner to connect to existing liner |
| O-16 to O-12 | CIPP Liner, CIPP Lateral Repairs |

**TABLE 7.2.14.2 – ALDER WAY, COST ESTIMATE**

| Alder Way Project #N | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $10,000.00 | $10,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $7,500.00 | $7,500.00 |
| 3 | 8" CIPP Liner | lf | 1110 | $40.00 | $44,400.00 |
| 4 | New shallow manholes | ea | 1 | $4,500.00 | $4,500.00 |
| 5 | CIPP Lateral Liners | ea | 22 | $2,500.00 | $55,000.00 |
| | | Construction Total | | | $121,400.00 |
| | | Contingency (25%) | | | $31,000.00 |
| | | Subtotal | | | $152,400.00 |
| | | Engineering (20%) | | | $30,500.00 |
| | | Administrative Costs (3%) | | | $4,600.00 |
| | | Total Project Costs | | | $187,500.00 |


**MAP 7.3.14 ALDER WAY PROJECT N**

### 7.3.15 Manhole Rehabilitation Project O

A project has been created to repair manholes found to be leaking during smoke testing and flowmapping reports. The City's manholes are very old and in poor shape in many locations due to the high proportion of older developments. The City has a limited capability to repair some of these manholes but for manholes with significant damage a specialized repair company should be contracted to perform a more permanent fix.

The manhole rehabilitation list was created from the information on the City's mapping. However this mapping is only approximate and some manhole locations do not exist or are not located where depicted. Effort was made to identify as closely as possible each manhole location and visually identify leaks or cracks in the subsurface structure.

Assumptions made in the cost portion included; filling a void at each manhole, average 8 foot manhole depth, sealing the manhole bench and all rings joints to the top rim, and sealing all cracks inside the manhole riser sections sufficient to pass a vacuum test.

Investigative surveys did not note any extensive hydrogen sulfide damage. This likely due to the steep slopes facilitating rapid water movement and little detention time. It may not be necessary to epoxy coat any of the manholes and this should be evaluated during the engineering process. Our recommendation is to use urethane foam to fill voids and to use fiber-reinforced mortar for joints and crack sealing.

**TABLE 7.2.15 – MANHOLE REHAB, COST ESTIMATE**

| Manhole Rehab Project #O | | | | | |
|---|---|---|---|---|---|
| Item No. | Description | Units | Quantity | Unit Cost | Total Cost |
| 1 | Mobilization Costs | ls | 1 | $5,000.00 | $5,000.00 |
| 2 | Construction and Temporary Facilities | ls | 1 | $3,500.00 | $3,500.00 |
| 3 | Manhole Sealing (30) | lf | 240 | $175.00 | $42,000.00 |
| 4 | Manhole void filling | ea | 30 | $100.00 | $3,000.00 |
| | | Construction Total | | | $53,500.00 |
| | | Contingency (25%) | | | $14,000.00 |
| | | Subtotal | | | $67,500.00 |
| | | Engineering (20%) | | | $13,500.00 |
| | | Administrative Costs (3%) | | | $2,100.00 |
| | | Total Project Costs | | | $83,100.00 |

# 8.0 Capital Improvement Plan and Financing Options

*Section*
**8**

## *8.1 Introduction*

This section describes the prioritization of improvement projects developed in Section 7 and their associated costs. Projects have been grouped into priority levels based upon relative pipe condition and their I/I burden upon the collection system.

All of the improvement projects were assigned priority levels based upon a combination of objective and subjective factors. Objective factors included:

- Visible sinkholes in the pavement
- Broken pipe chunks lying inside the pipe
- Abnormally high flow measurements
- Visible pipe bellies or surcharged manholes.

Subjective factors included:

- Comments from system operators of known problems
- Judgment of the condition of pipe walls and manhole rings from good to poor
- Observation of high flow lines in pipe
- Estimation of the root causes of grease and sediment buildup.

Projects and priorities are based upon information gained from the three investigative surveys. Each survey was performed in a manner to cost effectively determine the most significant deficiencies throughout the system. As the surveys cannot provide perfect information about the entire collection system, it is possible other urgent failures or deficiencies may become evident before the projects are complete.

Development of each project included selection of an appropriate repair technique and analysis of additional costs for each area. Many of the projects have trenchless repair methods initially recommended based upon the analysis of televised data. During design, this televised data must be coordinated with relevant construction firms to verify the applicability of each proposed repair method or other mitigating cost factors. Open trench projects may come upon unidentified buried obstacles or poor soil conditions. Therefore, when estimating projects, a 25% contingency was planned at this preliminary planning stage to account for all of these unknowns.

Table 8.1.1 includes the total of all the improvement projects. Priority levels and groupings are discussed in the following sections.

**TABLE 8.1.1 – LIST OF REHABILITATION PROJECTS**

| Project | Project Number | Estimated Cost |
|---|---|---|
| Patching Project | A | $59,550.00 |
| N Nye St Replacement Project | B | $170,730.00 |
| NE 12th St Project | C | $137,530.00 |
| SE 10th St Project | D | $59,340.00 |
| E Graham St Project | E | $57,075.00 |
| NW 6th St Project | F1,F2 (F1 cost) | $107,295.00 |
| Business 20 Replacement Project | G | $189,290.00 |
| SE 5th St Project | H | $45,500.00 |
| SE Alder St Project | I | $34,160.00 |
| Butler Bridge Slope Project | J1, J2 (J1 cost) | $214,300.00 |
| N Main St Project | K | $32,420.00 |
| Business 20 Bursting Project | L | $48,110.00 |
| Alley Repair Project | M | $10,775.00 |
| Alder Way Project | N | $187,500.00 |
| Manhole Rehab Project | O | $83,100.00 |
| | **TOTAL** | **$1,436,675.00** |

The combined total for all the combined projects is **$1,436,675.00**

A scorecard combining the observations from the data in the Smoke Testing, Flow Mapping and Television Survey is shown in Table 8.1.2. Each survey is scored using the objective and subjective factors discussed earlier to rate the pipe segments. The Television Survey was given a higher weighting factor because it is precise and observes infiltration, inflow, pipe condition and grade concurrently.

The rankings in Table 8.1.2 are used to separate the fifteen rehabilitation projects into the four priority improvement plan projects.

**TABLE 8.1.2 REHABILITATION PROJECT SCORECARD**

| Project Name | Project Number | Average Smoke Testing Weighting (Pipe Segment Score/Total Total Segments) | Average Flow Mapping Weighting (Pipe Segment Flow Ranking/Total Pipe Segments) | Average Televise Weighting (Pipe Segment Televise Score/Total Pipe Segments) | Score (Smoke X 2+Flow X 2+Televise X 5)/3 | Rank |
|---|---|---|---|---|---|---|
| Patching Project | A | 0.8 | 0.6 | 1.2 | 2.9 | 15 |
| N Nye St Replacement Project | B | 1 | 3 | 3 | 7.7 | 4 |
| NE 12th St Project | C | 3 | 1 | 2.7 | 7.2 | 6 |
| SE 10th St Project | D | 3 | 0 | 4 | 8.7 | 3 |
| E Graham St Project | E | 1 | 3 | 3 | 7.7 | 5 |
| NW 6th St Project | F | 3 | 0 | 4 | 8.7 | 2 |
| Business 20 Replacement Project | G | 0 | 3 | 3 | 7.0 | 8 |
| SE 5th St Project | H | 2 | 0 | 3 | 6.3 | 10 |
| SE Alder St Project | I | 2 | 0 | 2.7 | 5.8 | 12 |
| Butler Bridge Slope Project | J | 3 | NA | 4 | 13.0 | *1 |
| N Main St Project | K | 2 | 0 | 2.5 | 5.5 | 13 |
| Business 20 Bursting Project | L | 1 | 1 | 2 | 4.7 | 14 |
| Alley Repair Project | M | 3 | 0 | 3 | 7.0 | 7 |
| Alder Way Project | N | 1 | 0 | 3 | 5.7 | 11 |
| Manhole Rehab Project | O | 3 | 3.5 | NA | 6.5 | *10 |

Smoketesting results rated from 0-3, 3 being highest inflow and 0 being no smoke returns
Flowmapping results rated from 0-3, 3 being very high infiltration and 0 being none measured
Televising rated from 0-4, using ratings shown in Appendix A
Data averaged between all pipe segments included in a project
*Unavailable data, score divided by 2 instead

## *8.2 Priority 1 Projects*

Priority 1 projects should be undertaken immediately. The pipe segments grouped as Priority 1 contain the significant deficiencies of the following types:

- Extreme root intrusion
- Many separated or offset pipe joints
- I/I throughout the pipe
- Significant concrete deterioration

At minimum all roots should be cut which will re-open the pipe access temporarily but possibly increase infiltration (the roots may be helping "plug" the leaks and their removal may increase the effective void size). Root cutting will temporarily reduce maintenance associated with clogged sewers. Design and planning of the replacement project for these pipelines should proceed regardless of the status of root cutting repairs. Included projects are listed in Table 8.2.

**TABLE 8.2 – PRIORITY 1 PROJECTS: INCLUDED REHABILITATION PROJECTS**

| Priority Ranking | Project # | Project Name | Project Cost |
|---|---|---|---|
| 1 | J1 | Butler Bridge Slope Project | $214,300.00 |
| 2 | F | NW 6$^{th}$ Street Project | $107,295.00 |
| 3 | D | SE 10$^{th}$ Street Project | $59,340.00 |
| | | **Total Priority 1 Projects** | **$380,935.00** |

## 8.3 Priority 2 Projects

Priority 2 projects deficiencies are similar in scope to those in Priority 1, but with diminished root intrusion. The pipe segments grouped as Priority 2 contain the significant deficiencies of the following types:

- Many leaking joints
- Broken pipe
- Holes in pipe
- Poor grade with standing water and offset joints
- Significant concrete deterioration

These projects should be started as soon as the Priority 2 projects are completed, or in the next 3-4 years. Included projects are listed in Table 8.3.

**TABLE 8.3 – PRIORITY 2 PROJECTS: INCLUDED REHABILITATION PROJECTS**

| Priority Ranking | Project # | Project Name | Project Cost |
|---|---|---|---|
| 4 | B | N Nye Street Replacement Project | $170,730.00 |
| 5 | E | E Graham St Project | $57,075.00 |
| 6 | C | NE 12$^{th}$ Street Project | $137,530.00 |
| 7 | M | Alley Repair Project | $10,775.00 |
| 8 | G | Business 20 Project | $189,290.00 |
| | | **Total Priority 2 Projects** | **$565,400.00** |

## 8.4 Priority 3 Projects

Priority 3 projects are in significantly better condition than Priority 1 and 2 projects. Rehabilitation of this project group is targeted towards I/I reduction and less towards structural and maintenance deficiencies. Repairs typically required in Priority 3 include:

- Isolated leaking joints
- Cracks or holes in pipe
- Lateral to mainline joint separation
- Concrete deterioration

Priority 3 projects should be completed in the next 5-6 years. Included projects are listed in Table 8.4.

**TABLE 8.4 – PRIORITY 3 PROJECTS: INCLUDED REHABILITATION PROJECTS**

| Priority Ranking | Project # | Project Name | Project Cost |
|---|---|---|---:|
| 9 | P | Manhole Rehab Project | $83,100.00 |
| 10 | H | SE 5th Street Project | $45,500.00 |
| 11 | N | Alder Way Project | $187,500.00 |
| 12 | I | SE Alder St Project | $34,160.00 |
| | | **Total Priority 3 Projects** | **$350,260.00** |

## 8.5 *Priority 4 Projects*

Priority 4 projects are strictly I/I repair projects where the pipe sections are in reasonable condition. The North Main Street and Business 20 Bursting Projects are to repair average condition concrete pipe containing a moderate amount of infiltration points. The Patching Project is a bundle of projects needing point repairs to eliminate smaller I/I sources.

Any of these projects are potentially good candidates to combine with other similar repair methods in Priorities 1-3, or could be repaired together at a future date. Priority 4 projects should be completed in the next 10 years. Included projects are listed in Table 8.5.

**TABLE 8.5 – PRIORITY 3 PROJECTS: INCLUDED REHABILITATION PROJECTS**

| Priority Ranking | Project # | Project Name | Project Cost |
|---|---|---|---:|
| 13 | K | N Main Street Project | $32,420.00 |
| 14 | L | Business 20 Bursting Project | $48,110.00 |
| 15 | A | Patching Project | $59,550.00 |
| | | **Total Priority 4 Projects** | **$140,080.00** |

## 8.6 *Funding Options*

Repairs to the collection system can be funded in a variety of ways. State and Federal programs provide low interest loans and grants to municipal wastewater systems. The City can provide its own funding through current or future revenues. There also is the option of issuing local bonds to pay for immediate improvements and finance them over a fixed term.

The City is already faced with substantial upgrades and plans repairs for the potable water system. Therefore, the City is tasked with raising a sizeable amount of funds to complete the rehabilitation projects we have recommended. The major funding sources will be briefly discussed in the following paragraphs. The State of Oregon holds "One Stop" meetings monthly in Salem where the City can schedule a time to learn about all the current Federal and State program offerings.

### 8.6.1 State Funding Sources

Oregon DEQ administers a loan program on behalf of the EPA. The *Clean Water State Revolving Fund* (CWSRF) Loan Program provides low-cost loans for the planning, design and construction of various water pollution control activities. It provides a subsidized loan package for planning, design, construction, emergencies, urgent repairs and local community projects. Rates currently vary from 1.09% to 4.35% depending on the project type. Loan terms 5 years and greater include a 0.5% annual fee for administration.

The Oregon Infrastructure Finance Authority (IFA) provides low cost loans for projects up to $9 million in size. Loan terms are offered up to 25 years of the life of the project and come from a dedicated public works fund.

The IFA also offers a water/wastewater loan fund with similar terms. These loans are typically paid through bonding.

Another program offered by the IFA is a grant program. The grant program is targeted toward disadvantaged income areas and has a $1 million cap for wastewater projects. The IFA states 1 of 3 criteria must be met for eligibility:

1. The proposed activities must benefit low- and moderate-income individuals.
2. The activities must aid in the prevention or elimination of slums or blight.
3. There must be an urgent need that poses a serious and immediate threat to the health or welfare of the community.

Other grant caps and information can be found by visiting the IFA website http://www.orinfrastructure.org/

### 8.6.2 Federal Funding Sources

Many of the Federal Funds are administered through the DEQ and IFA programs. The major source of direct federal funding for communities comes from the U.S. Department of Agriculture (USDA). The USDA administers the Rural Development (RD) program which provides funding through the Rural Utilities Service (RUS).

Loans and Grants are both available under the RUS program. Grants from both RUS and the state IFA programs both contain revenue guidelines that favor sanitary districts set at already high rates. Because Toledo is a smaller community it is eligible for these grants. Federal funds have specific additional requirements and steps which must be taken throughout the design and construction process. The City will need to weigh the additional costs against the size of benefits they are receiving to ultimately make a decision.

### 8.6.3 Revenue Sources

Revenue funding originates directly from rate payers within the City's. Rate increases are not popular with residents, especially those on fixed incomes, but are often necessary to provide funding for loan and bond payments or to save up for future repairs. Revenue rates are also often raised to meet minimum guidelines for State or Federal financing sources. Government funding agency guidelines are set to ensure districts are not charging unreasonably low rates to maintain the system before they offer financial assistance.

The City should evaluate its rate structure and see how the rates compare with other like size cities. Many coastal cities and sanitary districts have recently gone through this process to align their rate structure with the maintenance needs of their systems.

### 8.6.4 Bonds

Bonds come in two different varieties, general obligation bonds and revenue bonds. The City would issue a bond to pay for the project(s) and pay the bond and interest back over a fixed term. Bonds can be issued from 1 to 30 years in duration. Recommended practice is to avoid bonding beyond the life expectancy of the project. Wastewater facilities have a planning life expectancy of 20 years, although new manholes and sewer pipe commonly are expected to last beyond 50 years.

General obligation bonds are backed by a temporary property tax assessment and would raise taxes for users within the sanitary service area until the end of the bond term. General obligation bonds typically carry a lower interest rate as the property owners are under threat of foreclosure if taxes are not paid.

Revenue bonds set aside a portion of the user fees for sanitary sewer service and use those to repay the bond and interest. They do not result in an increase of taxes on the users and are typically regarded as riskier bonds with a slightly higher interest rate.

Due to the current economic conditions both general obligation and revenue bonds currently carry very low interest rates. Rates for municipal bonds are ranging from approximately 1.25% annually for a 5 year to 4.2% for a 30 year bond. The exact rate varies depending on the credit rating of the City and investor demand for the bonds.

# APPENDIX A

# Video Inspection Notes

| Repair Urgency | Color | Weighting Factor |
|---|---|---|
| No Repair or Small Repair | | 0-1 |
| Further Inspection or Repair | | Varies |
| Moderate Repair | | 2 |
| Extensive Repair | | 3 |
| Immediate Repair | | 4 |

| PIPE AND COMMENTS (MH TO MH) | | LINEAR FOOTAGE LOCATION |
|---|---|---|
| C5 to C6 | | |
| Crack with Deposits | | 78' |
| Pipe Belly | | 125' to 139' |
| Pipe Belly | | 231' to 242' |
| Overall pipe looks in good condition for Concrete Pipe | | |
| | | 373.84' |
| C21 to C18 | | |
| Leaking joint at manhole C18 | | 65' |
| Overall pipe looks in good condition for Concrete Pipe | | |
| | | 65.01' |
| | | |
| B29 to B31 | | |
| Leaking along pipe wall | | 10' |
| Large hole near bottom with I/I | | 31.5' |
| Small hole near bottom of pipe | | 82.5' |
| Large I/I at lateral connection | | 136' |
| ABS pipe patch at | | 148' |
| Lateral with sizeable clear flow | | 170' |
| Lateral with small leak around penetration | | 299' |
| Joint looks rough | | 318' |
| Joint looks rough | | 324' |
| Pipe begins to look rougher | | 329' |
| Joint looks rough | | 338' |
| Small hole near bottom of pipe | | 354' |
| Large hole near bottom with I/I | | 357' |
| Pipe begins to look smoother | | 360' |
| Lateral has high flow, joint appears poor | | 395' |
| Capped lateral leaking | | 409' |
| Lateral has high flow | | 412' |
| Lateral has high flow, joint appears poor | | 455' |

| | | |
|---|---|---|
| Pipe in average condition, some spot repair or section repairs acceptable | | |
| | | 463.25' |
| | | |
| B20 to B18 | | |
| Large Roots | | 6' |
| Large Roots | | 9' |
| Long Crack and Roots | | 31' |
| > 30 wet looking spots | | 42' to 200' |
| Rough Joint possible leak | | 51' |
| Pipe rough at top | | 66' to 73' |
| High Lateral flow | | 102' |
| Roots on bottom | | 138' |
| Ring cracks | | 161' |
| Large Hole | | 164' |
| Roots | | 178' |
| Roots | | 193' to 195' |
| Pipe in poor condition, needs complete repair | | |
| | | 218.59' |
| | | |
| B22 to B20 | | |
| Pipe rough at lateral | | 15' to 17' |
| Pipe rough | | 40' |
| Pipe Wet | | 56' |
| Pipe Pinhole Leak | | 67' |
| Small hole | | 70' |
| Possible Ring Crack | | 94' |
| Possible Ring Crack | | 98' |
| Pipe in average condition, a few small repairs possible | | |
| | | 119.13' |
| | | |
| B16 to B18 | | |
| Pipe has complete belly | | |
| Pipe in poor condition, no specific repair areas noted due to belly | | |
| | | 46.29' |
| | | |
| B16 to B12 | | |
| Wet | | 6' |
| Small hole | | 9' |
| Small hole | | 109' |
| Lateral high flow | | 144' |
| Overall pipe looks in good condition for Concrete Pipe | | |

| | | |
|---|---|---|
| | | 243.97' |
| | | |
| O7 to O6 | | |
| Lateral stopped video at 223.53' | | |
| Pipe is very rough and worn, likely flowing full often, no issues seen | | |
| | | 223.53' |
| | | |
| N3 to N4 | | |
| Deposit Buildup | | 7' to 12' |
| Roots Light | | 42' to 59' |
| Roots Heavy | | 59' to 165' |
| Leak | | 75' |
| Roots Light | | 175' to 191' |
| Deposit Buildup | | 199' |
| Roots Light | | 246' to 291' |
| Pipe in very bad condition, quick replacement suggested | | |
| | | 291.13' |
| | | |
| N4A to N4 | | |
| Pipe in average condition, no repairs needed | | |
| | | 141.21' |
| B1 to F41 | | |
| High Lateral Flow | | 104' |
| High Lateral Flow | | 107' |
| Very High Lateral Flow | | 242' |
| Pipe in good condition, laterals need inspected | | |
| | | 328.42' |
| | | |
| F41 to F38 | | |
| High Lateral Flow | | 104' |
| Large Belly going into manhole | | 200' |
| Pipe in good condition except belly | | |
| | | 200' |
| | | |
| B9 to B1 | | |
| Pipe in good condition | | |
| | | 117.75' |
| | | |
| F38 to F36 | | |
| Pipe in good condition | | |
| | | 126' |

| | | |
|---|---|---|
| | | |
| F36 to F34 | | |
| Pipe in good condition | | |
| | | 130.26' |
| | | |
| F34 to F33 | | bAd Video |
| | | |
| F9 to F8 | | |
| Pipe in good condition | | |
| | | 398.64' |
| | | |
| I33A to I33 | | |
| Pipe in good condition | | |
| | | 21.31' |
| | | |
| I33A to I4 | | |
| Pipe in good condition | | |
| | | 185.27' |
| | | |
| I34 to I33 | | |
| Small Roots | | 124' |
| Small Roots | | 132' |
| Small Roots at lateral | | 133' |
| Roots | | 134' |
| Roots | | 136' |
| Small Roots | | 141' |
| Small Roots | | 146' |
| Long Crack top of pipe | | 222' |
| Long Crack top of pipe | | 227' |
| Crack top of pipe | | 246' |
| Pipe in Average condition, problems are located in clusters | | |
| | | 280.69' |
| | | |
| I71A to I71 | | |
| Pipe in good condition | | |
| | | 20.05' |
| | | |
| I71 to I70 | | |
| Pipe in good condition | | |
| | | 223.24' |
| | | |

| | | |
|---|---|---|
| I23 to I84 | | |
| Roots or Gasket | | 41' |
| Holes in top of pipe | | 158' |
| Small Roots | | 164' |
| Small Roots | | 171' |
| Broken Joint | | 385' |
| Pipe in good condition, spot repairs advisable | | |
| | | 390.02' |
| | | |
| I72 to I71 | | |
| Pipe in good condition | | |
| | | 187.21' |
| | | |
| F26 to F23 | | |
| Large Root throughout pipe | | |
| Pipe in very bad condition, quick replacement suggested | | |
| | | 23.53' |
| | | |
| D1 to F8 | | |
| Leaking Joint | | 17' |
| Belly cannot see pipe | | 30' to 90' |
| Leaking Joint | | 116' |
| Offset Pipe | | 117' |
| Pipe in good condition, spot repairs advisable | | |
| | | 185.08' |
| | | |
| D1 to D2 | | |
| Pipe looks good but submerged 15' to end | | |
| | | 174.43' |
| | | |
| D2 to D3 | | |
| Submerged to 84' | | 84' |
| Submerged again at 115' to 124' | | 115' |
| Small section of pipe visible looks good | | |
| | | 124.26' |
| | | |
| D3 to D4 | | |
| Nearly Every joint in pipe is leaking | | |
| Belly | | 64' to 116' |
| Leak | | 119' |
| Pipe in poor condition and should be lined or replaced | | |

| | | |
|---|---|---|
| | | 205.42' |
| | | |
| K29 to K28 | | |
| Wide Joint | | 30' |
| Pipe begins to look very worn | | 37' |
| Extremely worn pipe | | 100' to 103' |
| Deposits in pipe | | 154' |
| Deposits in pipe | | 161' |
| First Roots in pipe | | 164' |
| Roots become worse | | 168' |
| End of Roots in pipe | | 179' |
| Small Roots | | 193' |
| Small Roots | | 195' |
| Small Roots | | 238' |
| Small Roots Begin | | 248' |
| Small Roots End | | 261' |
| Large Roots begin | | 270' |
| Hole in top of pipe | | 271' |
| Large roots end | | 278' |
| Large deposit or roots blocking camera | | 294' |
| Pipe in poor condition throughout | | |
| | | 296.75' |
| | | |
| I19 to I18 | | |
| Pipe good condition PVC to 172' | | |
| Concrete hole patch at | | 193' |
| Hole in Lateral top | | 247' |
| Pipe in good condition with 1 hole to patch | | |
| | | 365.03' |
| | | |
| I29 to I28 | | |
| Pipe in rough condition | | |
| | | 56.75' |
| | | |
| I28 to I29 rest of pipe | | |
| Big hole | | 84' |
| Pipe looks much less worn than upstream section | | |
| | | 122.52' |
| | | |
| I28 to I27 | | |
| Very Rough spot | | 46' |

| | | |
|---|---|---|
| Roots | | 54' |
| Roots | | 56' |
| Small Roots | | 57' |
| Small Roots | | 60' |
| Small Roots | | 66' |
| Roots | | 69' |
| Small Roots | | 74' |
| Leak | | 82' |
| Pipe in Average condition, downstream needs repaired | | |
| | | 94.24' |
| | | |
| I27 to I26 | | |
| Pipe in good condition | | |
| | | 122.03' |
| | | |
| K37 to K38 | | |
| Concrete pipe in average condition | | |
| | | 132.59' |
| | | |
| K38 to K39 | | |
| Concrete pipe in average condition | | |
| | | 99.17' |
| | | |
| K16 to K17 | | |
| Pipe wall look worn | | |
| Huge pipe offset | | 11' |
| Cannot video to cleanout | | |
| | | 11.33' |
| | | |
| K16 to K15 | | |
| Small Roots | | 19' |
| Begin small roots | | 26' |
| Begin heavier roots | | 41' |
| PVC pipe patch | | 61' - 64' |
| Begin roots | | 64' |
| Begin Heavy Roots | | 68' |
| Begin Extreme roots | | 90' |
| Pipe Joint Drop | | 156' |
| Pipe in extremely bad condition, replace soon | | |
| Pipe downstream on hill not videoable, likely in same condition | | |
| | | 156.8' |

| | | |
|---|---|---|
| K26 to K25 | | |
| Pipe in good condition | | |
| | | 218.69' |
| | | |
| K25 to K23 | | |
| Pipe in good condition, roots in manhole K23 | | |
| | | 166.2' |
| | | |
| M18 to M13 | | |
| Deposits on bottom | | 172' |
| Pipe in good condition cannot see further | | |
| | | 172.01' |
| | | |
| I81 to I78 | | |
| Root or gasket at joint | | 90' |
| Capped lateral leaking | | 109' |
| Leaking joint | | 116' |
| Capped lateral leaking | | 116' |
| Broken pipe joint leaking | | 138' |
| Broken pipe joint leaking | | 141' |
| Pipe in average to poor condition, repair in at least sections | | |
| | | 154.09' |
| | | |
| F20 to F18 | | |
| Small Roots | | 9' |
| Small Roots | | 12' |
| Joint is wet | | 14' |
| Joint is wet | | 16' |
| Small Leak on Wall | | 18' |
| Leaking Joint | | 19' |
| Joint is wet | | 21' |
| Joint is wet | | 24' |
| Roots | | 26' |
| Small Roots | | 39' |
| Small Roots | | 56' |
| Roots | | 59' |
| Pipe extremely worn | | 63' |
| Lateral with roots | | 65' |
| Pipe becomes less worn | | 67' |
| Roots and wet joint | | 69' |

| | | |
|---|---|---|
| Small Roots | | 77' |
| Roots | | 79' |
| Roots | | 84' |
| Joint is wet | | 99' |
| Pipe is a mixture of average and poor sections | | |
| | | 103.15' |
| | | |
| D11 to D9 | | |
| Leaking joint | | 92' |
| Leaking joint | | 95' |
| Leaking joint | | 105' |
| Leaking joint | | 111' |
| Leaking joint | | 118' |
| Leaking joint | | 121' |
| Leaking joint | | 227' |
| Video missing | | 234 to 277 |
| Leaking joint | | 337' |
| Pipe in average condition, could use some joint repairs | | |
| | | 381.17' |
| | | |
| Clinic cleanout to F8 | | |
| Large belly at start | | |
| Pipe in good condition other than backwards wye connection | | |
| | | 208' |
| | | |
| F34 to F9 | | |
| Belly at 70' | | 70' to 74' |
| Lateral has high flow | | 177' |
| Pipe in good condition | | |
| | | 394.18' |
| | | |
| I69 to I74 | | |
| Capped Lateral leaking | | 73' |
| Leaking Joint | | 74' |
| Roots | | 119' |
| Roots | | 123' |
| Roots | | 128' |
| Roots | | 131' |
| Large concrete chucks in pipe | | 155' to 158' |
| Pipe in average condition, unknown where pipe sections come from | | |
| Suggest to repair pipe in specific areas | | |

| | | |
|---|---|---|
| | | 158.16' |
| | | |
| O16 to O12 | | |
| Leak in wall | | 155' |
| Leak in wall | | 222' |
| Pipe begins looking considerably worn | | 230' |
| Broken joint leaking | | 307' |
| Lateral with hole and large flow | | 368' |
| Pipe begins looking less worn | | 370' |
| Bad Leak at joint | | 395' |
| Pipe in average condition but well worn, some patching needed | | |
| | | 396.8' |
| | | |
| O12 to O7 | | |
| Small roots | | 12' |
| Small roots | | 18' |
| Leak around object portruding pipe | | 114' |
| Pipe in average condition, needs object removed | | |
| | | 116.12' |
| | | |
| O11 to O7 | | |
| Pipe appears well worn | | |
| Rat hole in lateral | | 30' |
| Lateral needs regrouted | | 101' |
| Roots growing around lateral | | 245' |
| Bottom broken out of pipe | | 266' |
| Roots growing around lateral | | 311' |
| Large hole in lateral joint | | 427' |
| Small roots | | 482' |
| Damage to joint | | 501' |
| Hole in lateral | | 518' |
| Gasket displaced | | 519' |
| Capped lateral with hole | | 564' |
| Leaking lateral | | 573' |
| Gasket displaced and pipe cracked | | 586' |
| Hole in lateral | | 613' |
| Hole in lateral and joint | | 638' |
| Pipe liner | | 664' to end |
| Pipe in poor condition except lined section | | |
| | | 738.1' |
| | | |

| | | |
|---|---|---|
| O11 to Cleanout | | |
| Lateral connection is bad, hole | | 5' |
| Many joints appear wet | | |
| Pipe and rock debris at end | | 48' |
| Pipe appears in average condition but joints possibly leaking | | |
| | | 48.72' |

# APPENDIX B

# Manhole Deficiency Notes

**TABLE B-1 – MANHOLE LEAKS FUOND IN FLOW MAPPING**

| Flow Mapping Manholes with Leaks<br>If strikeout shown City has repaired manhole & current condition listed to the right | |
|---|---|
| **Manhole #** | **Comments** |
| B10 | ~~Leaking~~ -OK |
| B16 | Leaking-Repaired but leaking still |
| B24 | Leaking |
| B27 | ~~Leaking~~-Fixed |
| C1 | Leaking |
| C2 | Leaking |
| D12 | ~~Leaking~~-Fixed |
| D4 | Leaking-Wet rings |
| D9 | Leaking-Repaired but leaking still |
| F15 | Leaking-Partially repaired, drill bit in wall |
| F8 | 10-20 GPM Leak-Still significant leaks |
| G33 | Bottom Ring Leaking-Repaired but leaking still |
| I4 | ~~Leaking~~-OK |
| L10 | Bottom Ring Leaking- Repaired but leaking still |
| L14 | ~~2 Leaks~~-Fixed |
| L15 | Leak Beside Lateral 1-2GPM-Repaired but leaking still |
| L8 | ~~Manhole Wet~~-OK |
| O12 | Bottom Ring Leaking |
| O5 | General Leaks- Bottom Ring |

## TABLE B-2 – MANHOLE LEAKS FOUND DURING SMOKE TESTING

**Smoke Testing Manholes with Improper Smoke Returns**

If strikeout shown City has repaired manhole & current condition listed to the right

| Manhole # | Comments |
|---|---|
| B32 | ~~Cracked manhole~~-Fixed |
| B76 | ~~Smoke beside manhole~~ Only around rim no leaking potential |
| B78 | ~~Smoke around rim~~-Ok just around rim no leaking potential |
| B78A | Leaking |
| B79 | ~~Smoke around rim~~-Just Rim Ok |
| C12 | ~~Smoke around rim –~~ No leak potential |
| C7 | ~~Cracked Manhole~~- No leak potential |
| C8 | ~~Cracked Manhole~~- No leak potential |
| D10 | ~~Cracked Rim~~-Only around rim no leaking potential |
| E1 | Smoke around rim – Cracked inside |
| F50 | ~~Smoke around rim~~- Only around rim no leaking potential |
| F51 | ~~Smoke from curb next to rim~~- Only around rim no leaking potential |
| F54 | ~~Smoke around rim~~ - Only around rim no leaking potential |
| F55 | ~~Smoke around rim~~ - Only around rim no leaking potential |
| G24 | ~~Smoke from manhole side~~ - Only around rim no leaking potential |
| H26 | Leaking around edges –Follow up as well |
| H27 | Leaking around edges–Follow up as well |
| H28 | Leaking around edges–Follow up as well |
| H32 | Broken Manhole in field–Follow up as well |
| H33 | Broken Manhole in field–Follow up as well |
| I31 | ~~Smoke around rim~~-OK |
| J1 | ~~Manhole cracked~~ |
| J2 | ~~Manhole cracked~~ |
| J3 | Smoke from ground – Leaking actively |
| K2 | Smoke coming from ground, replace with project |
| K25 | Cracked Manhole, large hole in top but no I/I risk |
| K33 | Smoke coming from ground –sinkhole nearby |
| K35 | Smoke around rim – Cannot find follow up |
| K37 | ~~Smoke from ground~~- Fixed |
| K6 | Leaking |
| K7 | Smoke around rim- Leaking |
| M38 | Smoke coming from ground-Mid ring leak |
| P19 | Smoke around rim-Grouted risers leaking |
| P32 | Smoke around rim-Many rings leaking |
| P5 | ~~Smoke from ground~~ –Not leaking, hole in ground |
| P9 | ~~Smoke from ground~~- OK |

APPENDIX C

# CITY OF TOLEDO

## INFLOW AND INFILTRATION STUDY

PROJECT NO.: 2902-008

March, 2011

# APPENDIX C

# BASIN AND SMOKE TESTING DRAWINGS

| | | | | |
|---|---|---|---|---|
| Civil West Engineering Services, Inc. | DWG BY: CDA<br>DATE: MACH, 2011 | 0 1" | **EXISTING COLLECTION SYSTEM** | |
| | I/I STUDY COVER SHEET | | CITY OF TOLEDO, COOS COUNTY, OREGON | COVER |

APPENDIX C

## LEGEND

- B-32 — MANHOLE AND NUMBER
- LEAKY MANHOLE
- REPAIRED MANHOLE
- GRAVITY SEWER

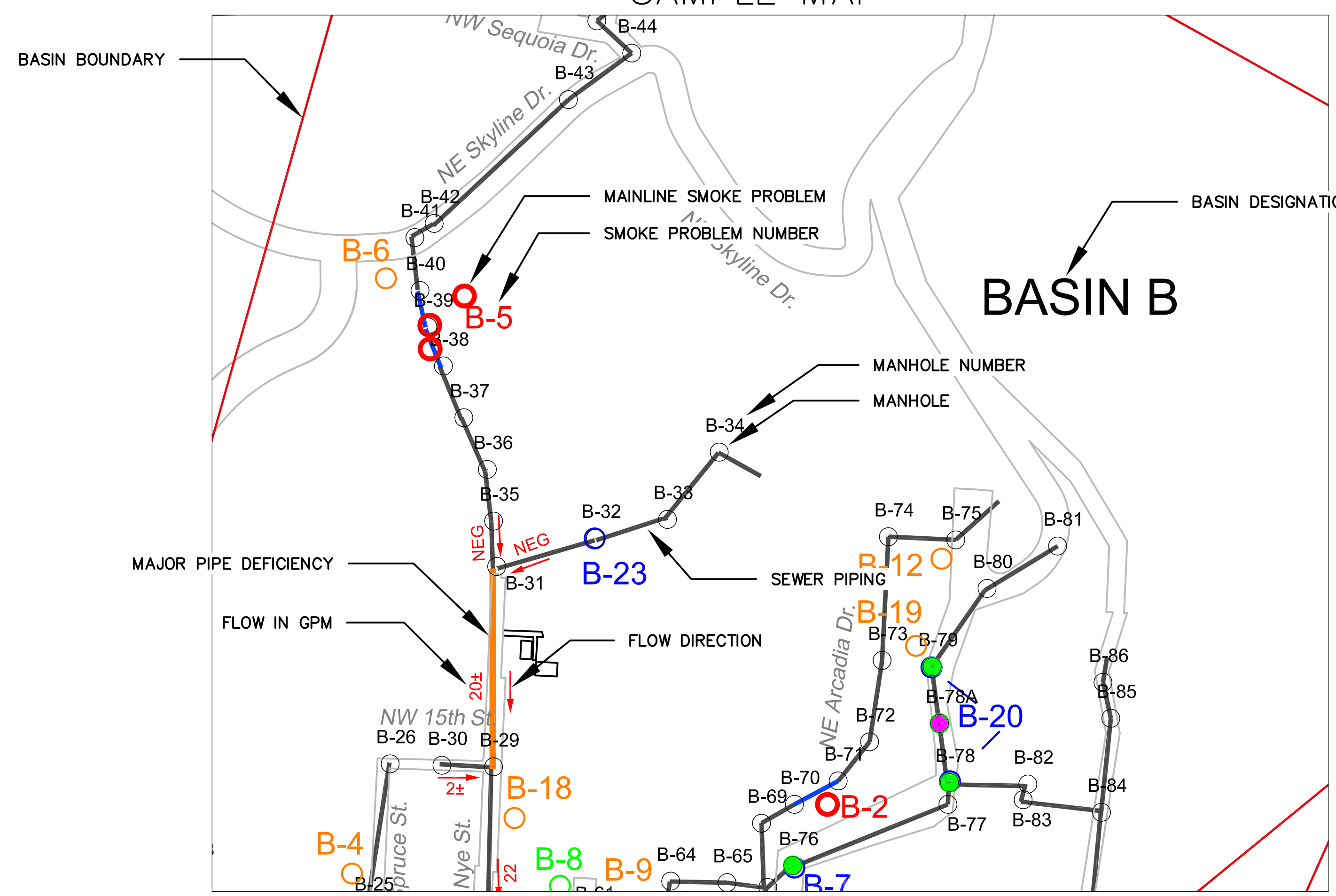
### SANITARY SEWER PIPE CONDITIONS

- EXTREME PIPE DEFICIENCY
- MAJOR PIPE DEFICIENCY
- MODERATE PIPE DEFICIENCY
- MINOR PIPE DEFICIENCY
- LATERAL CONNECTION DEFICIENCY
- FURTHER INSPECTION NEEDED

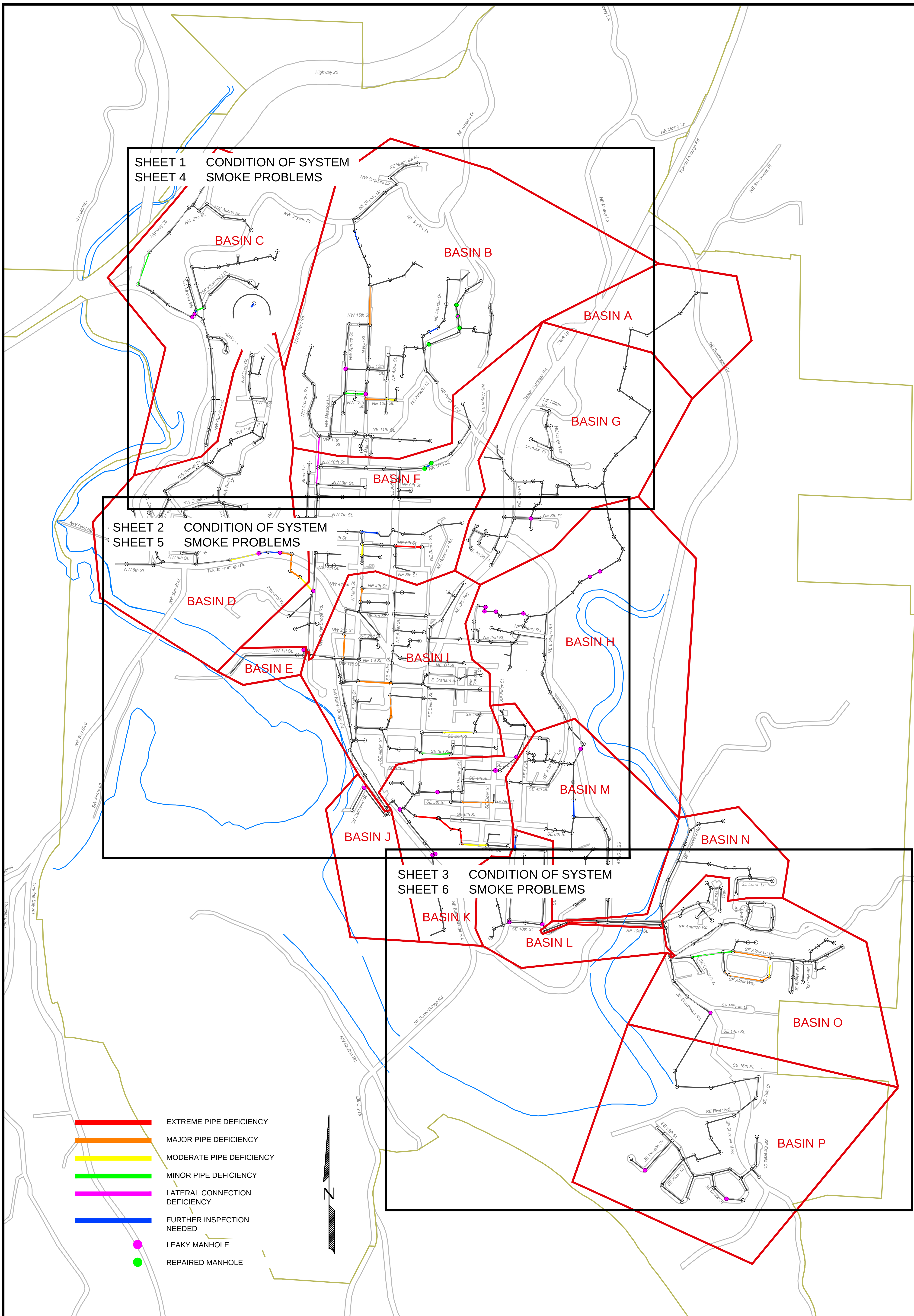
### SMOKE PROBLEMS

- V-1 — LATERAL SMOKE PROBLEM
- V-1 — SANITARY SEWER MAINLINE SMOKE PROBLEM
- V-1 — STORM DRAIN SMOKE PROBLEM
- V-1 — RESIDENTIAL SMOKE PROBLEM
- V-1 — MANHOLE SMOKE PROBLEM

## SAMPLE MAP




| | | |
|---|---|---|
| Civil West Engineering Services, Inc. | CITY OF TOLEDO<br>LINCOLN COUNTY, OREGON | EXISTING COLLECTION SYSTEM<br>LEGEND |
| 0 — 1" — 1" = 400FT<br>DRAWN BY: JBH<br>DATE: APRIL, 2010 | MAP<br>LEGEND | |




OVERVIEW

0 1"
1" = 1000'

DRAWN BY: JBH
DATE: MARCH, 2011

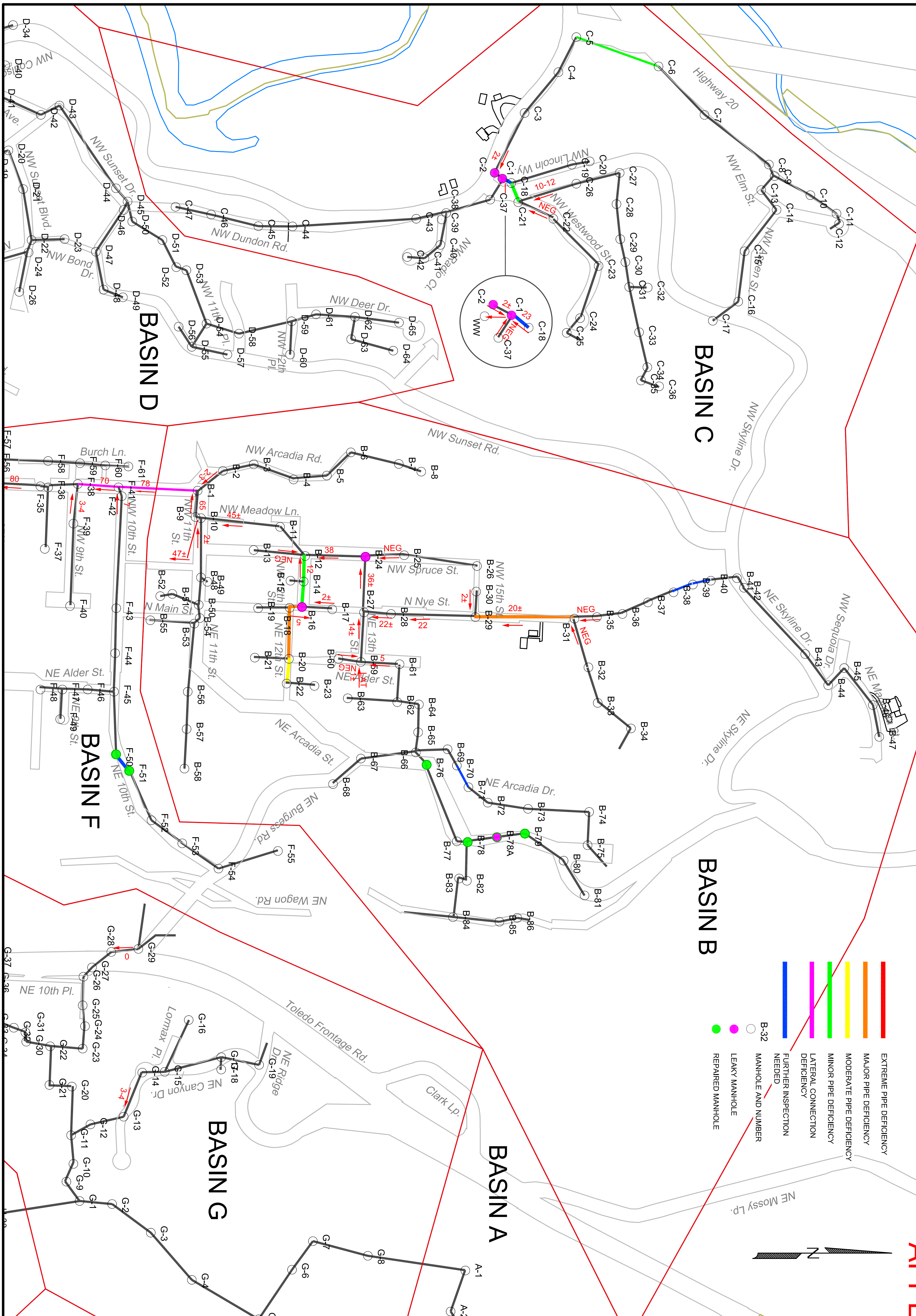
# EXISTING COLLECTION SYSTEM

OVERVIEW

CITY OF TOLEDO
LINCOLN COUNTY, OREGON

Civil West Engineering Services, Inc.

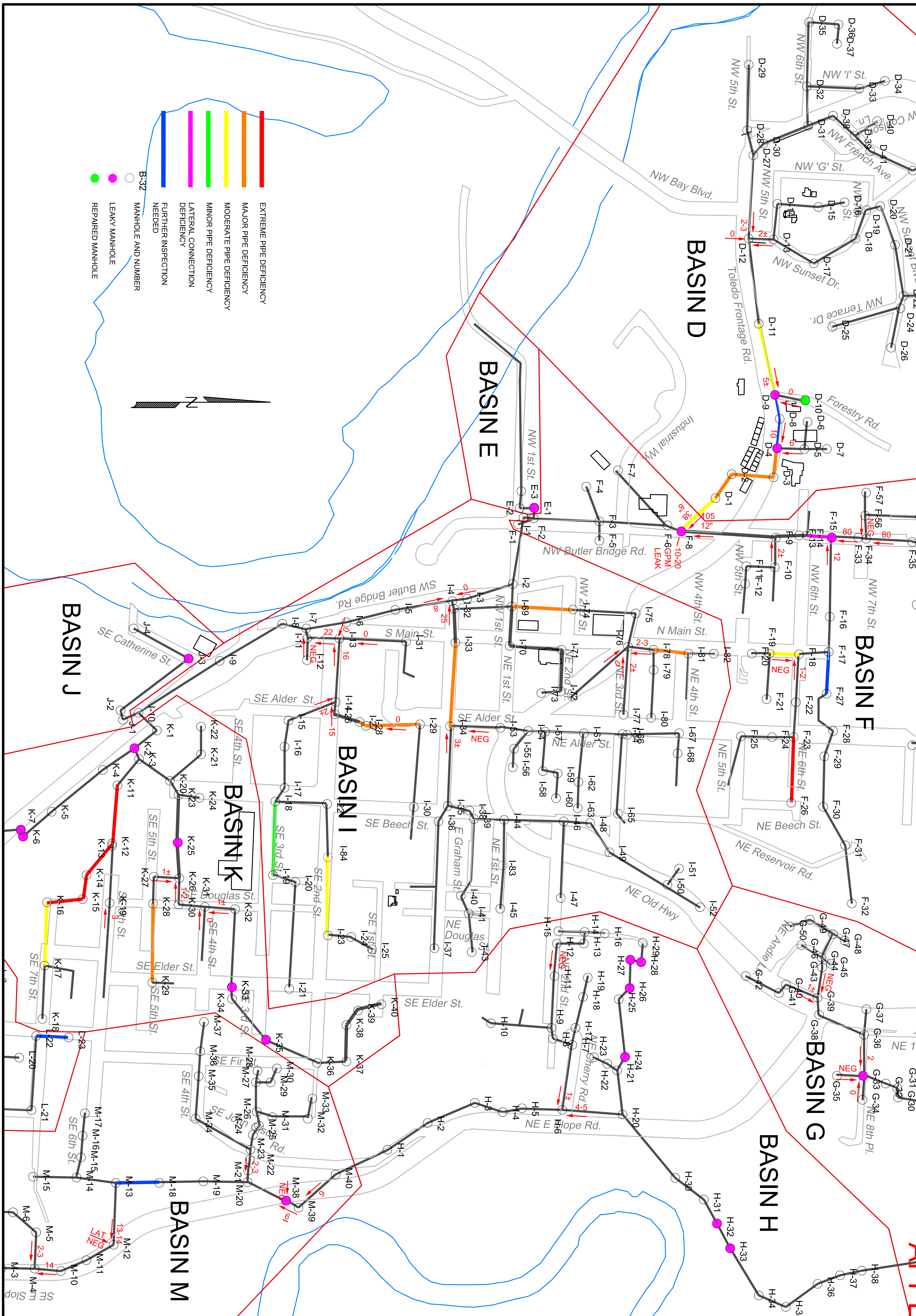
APPENDIX C



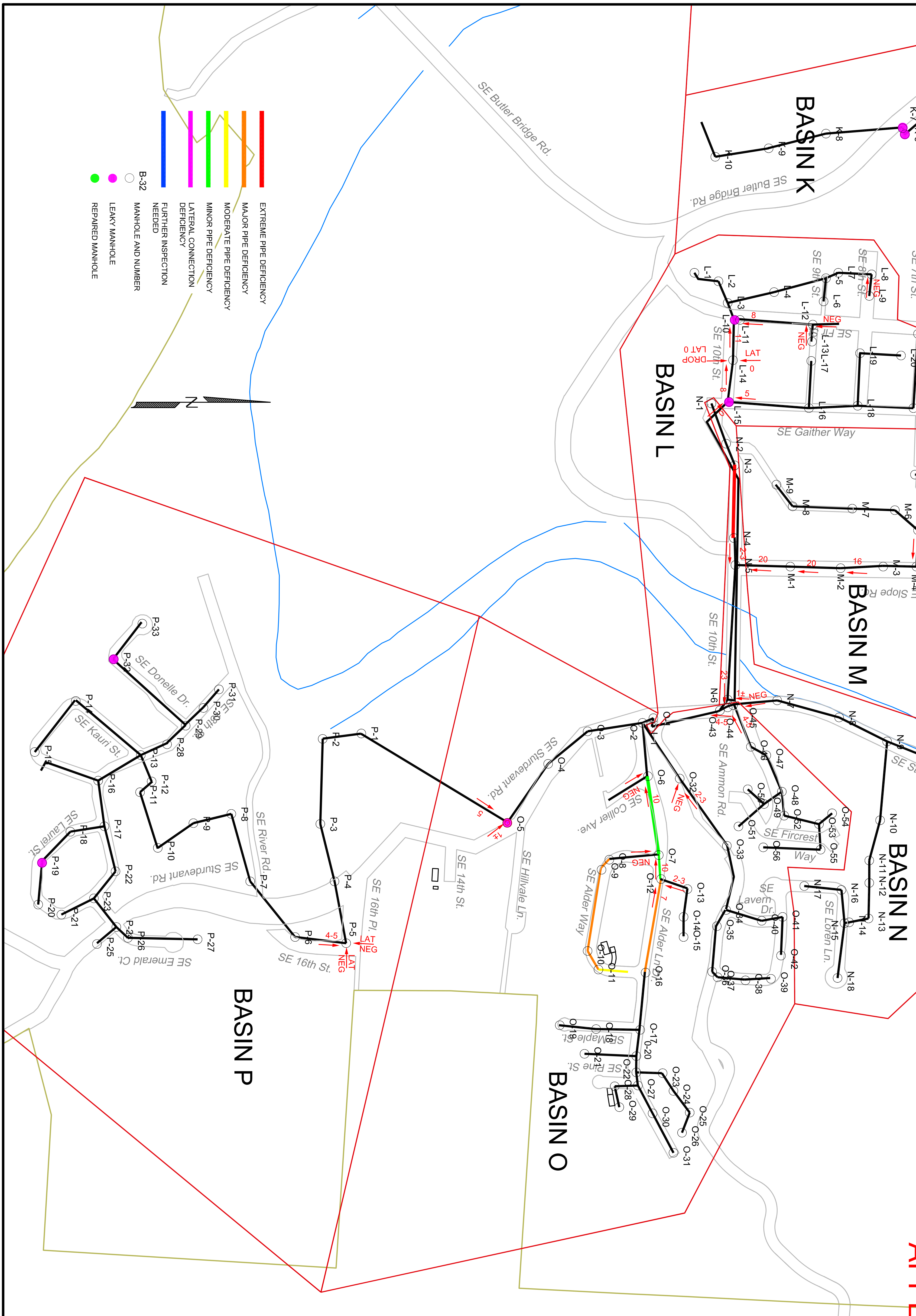

APPENDIX C

| | | | |
|---|---|---|---|
| SHEET 1 | 0 1"<br>1" = 400FT<br>DRAWN BY: JBH<br>DATE: MARCH, 2011 | EXISTING COLLECTION SYSTEM<br>CONDITION OF SYSTEM | CITY OF TOLEDO<br>LINCOLN COUNTY, OREGON |

Civil West Engineering Services, Inc.




| | | | | |
|---|---|---|---|---|
| SHEET 2 | 0 1" <br> 1" = 400FT <br> DRAWN BY: JBH <br> DATE: MARCH, 2011 | EXISTING COLLECTION SYSTEM <br> CONDITION OF SYSTEM | CITY OF TOLEDO <br> LINCOLN COUNTY, OREGON | Civil West Engineering Services, Inc. |




SHEET 3

0 1"
1" = 400FT

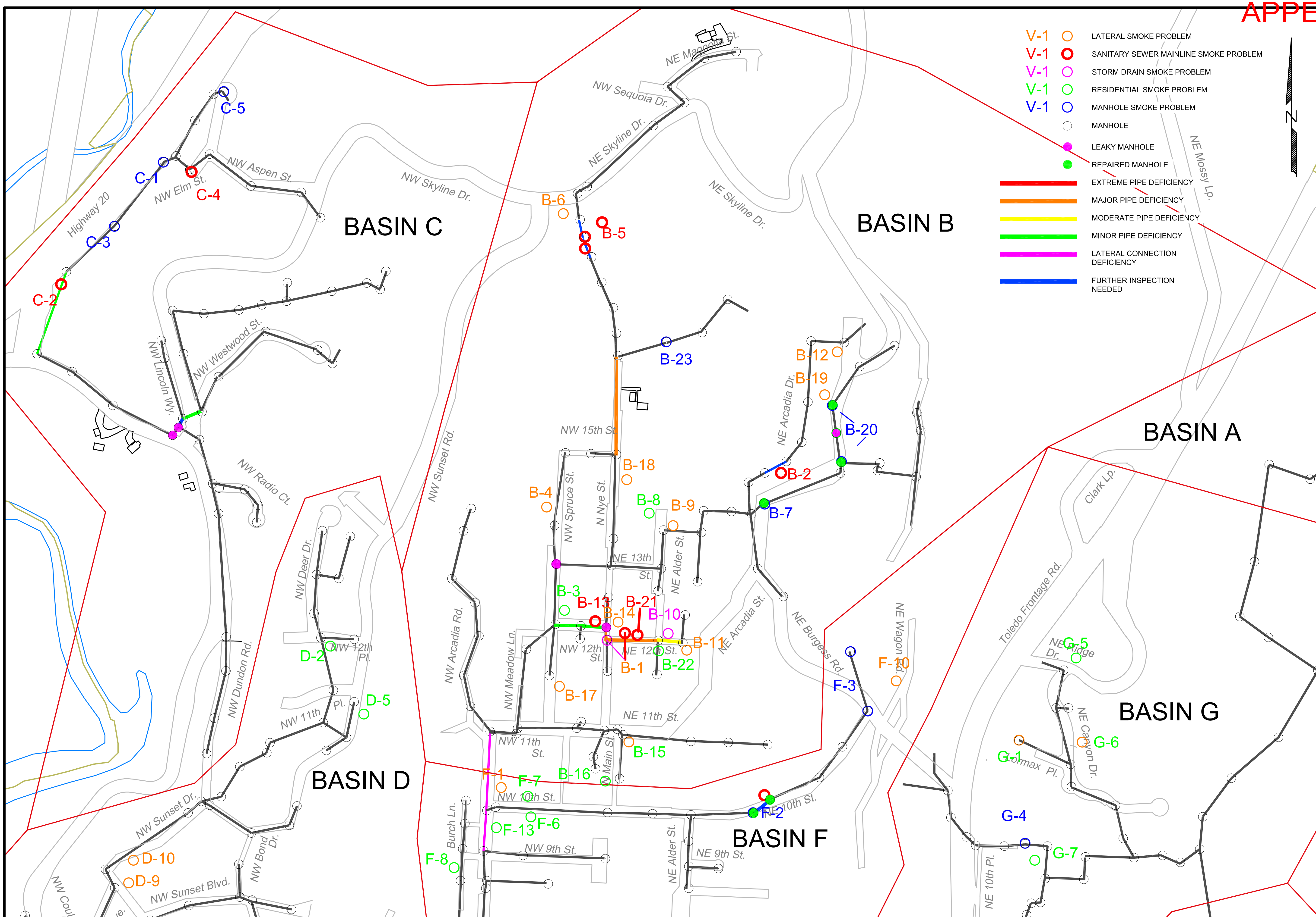
Drawn By: JBH
Date: March, 2011

EXISTING COLLECTION SYSTEM

CONDITION OF SYSTEM

City of Toledo
Lincoln County, Oregon

Civil West Engineering Services, Inc.




CITY OF TOLEDO
LINCOLN COUNTY, OREGON

EXISTING COLLECTION SYSTEM

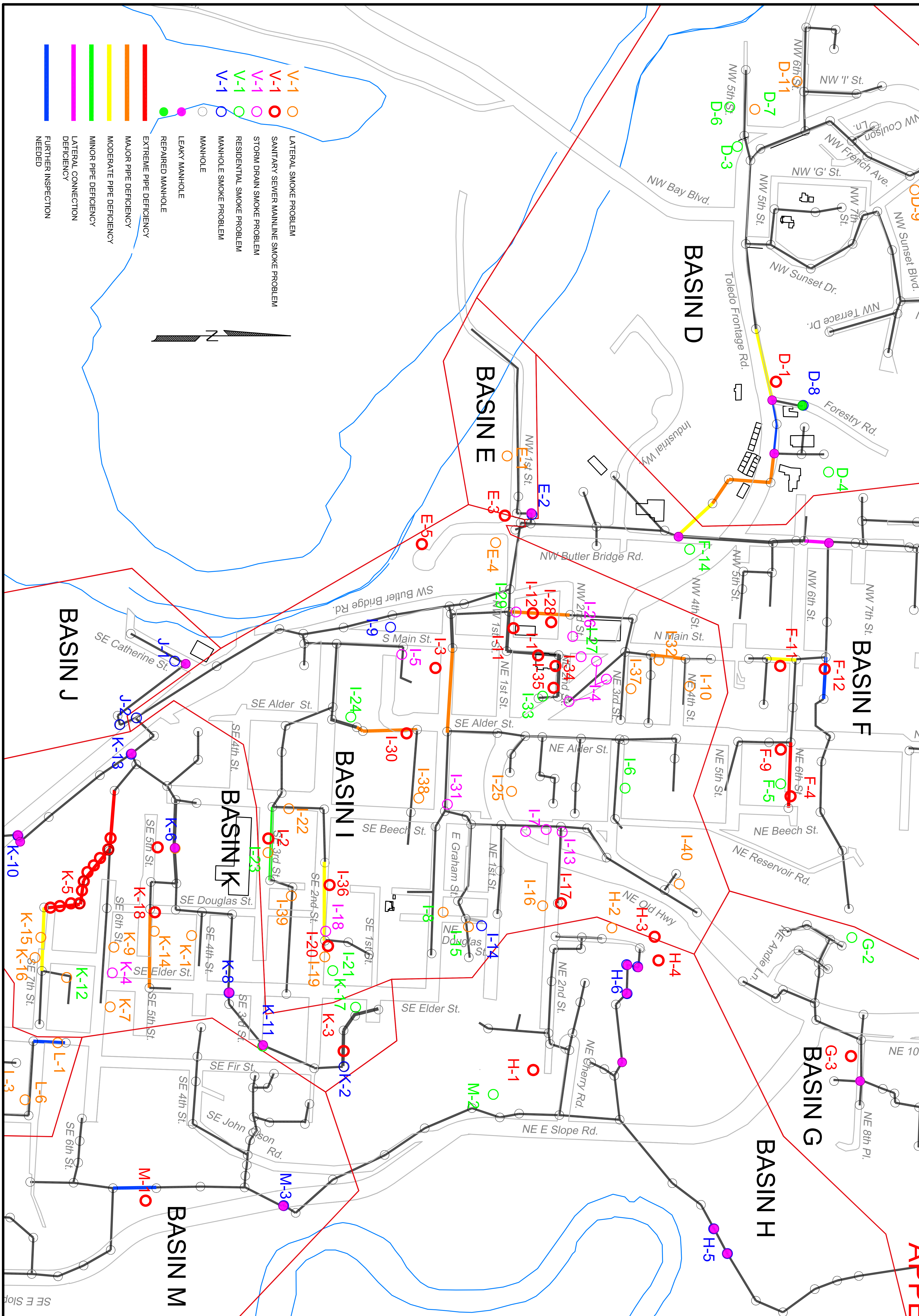
SMOKE PROBLEMS

0 1"
1" = 400FT

DRAWN BY: JBH
DATE: MARCH, 2011

SHEET 4

Civil West Engineering Services, Inc.




SHEET 5

0 1"
1" = 400FT

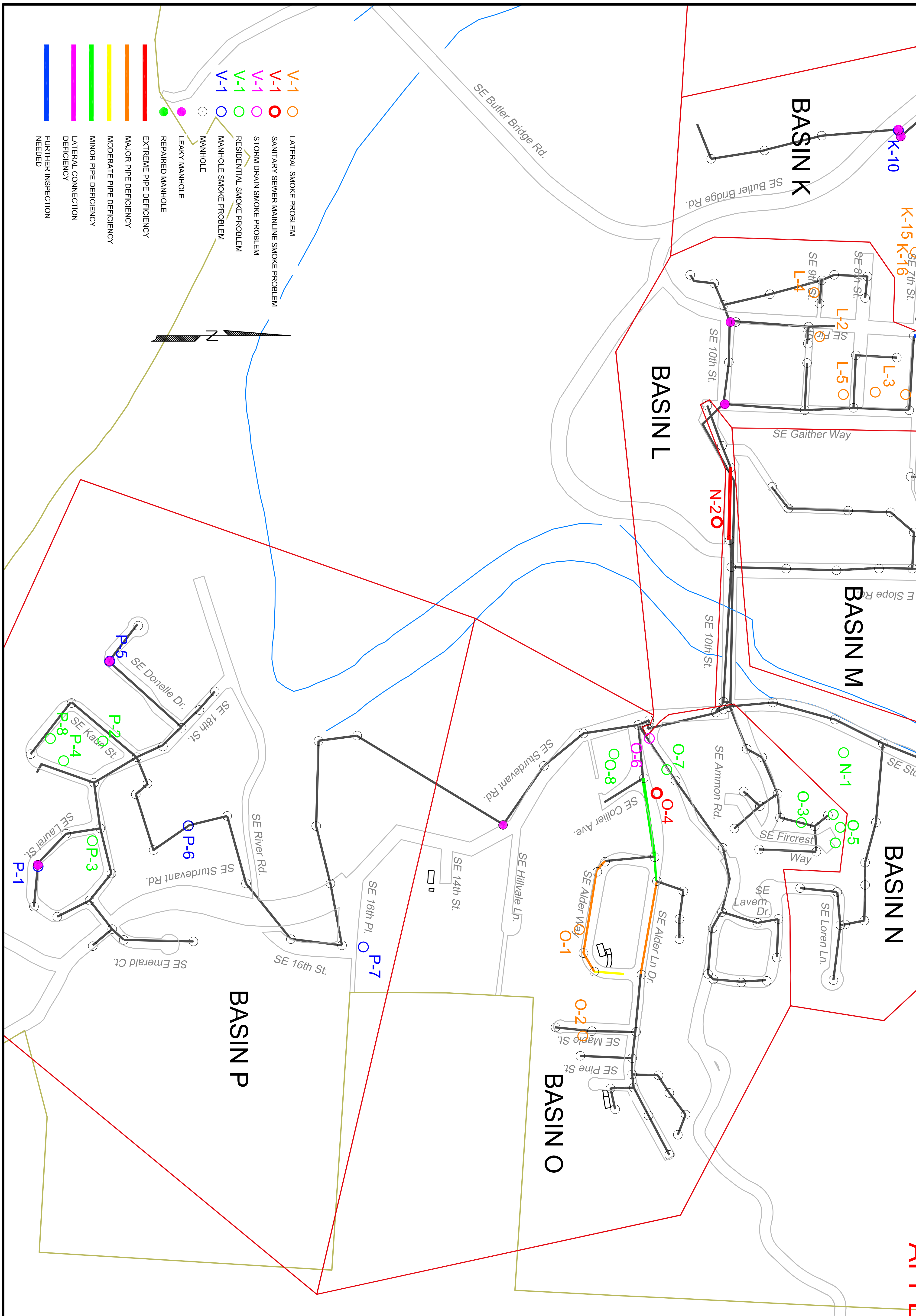
DRAWN BY: JBH
DATE: MARCH, 2011

EXISTING COLLECTION SYSTEM

SMOKE PROBLEMS

CITY OF TOLEDO
LINCOLN COUNTY, OREGON

Civil West Engineering Services, Inc.




APPENDIX C

| | | | |
|---|---|---|---|
| SHEET 6 | 0 1" <br> 1" = 400FT <br> DRAWN BY: JBH <br> DATE: MARCH, 2011 | EXISTING COLLECTION SYSTEM <br> SMOKE PROBLEMS | CITY OF TOLEDO <br> LINCOLN COUNTY, OREGON |

Civil West Engineering Services, Inc.

# APPENDIX D

**TABLE D-1 – LIST OF ALL DEFICIENCIES FOUND DURING SMOKE TESTING AS INDEXED IN BINDERS PROVIDED AT COMPLETION**

| | Type of Deficiency Observed on Smoketesting Report | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Report # | Residential Lateral | City Mainline | City Storm Drain | Residential Storm | Residential Downspout | Residential Plumbing | City Manhole | Residential Cleanout | Deficiency and Number of Each Deficiency Observed on Report Page |
| A1 | | | | | | | | 1 | |
| A2 | | | | | | | 1 | | |
| B1 | 1 | 1 | 1 | | | | | | |
| B2 | | 1 | | | | | | | |
| B3 | | | | | | | | 1 | |
| B4 | 1 | | | | | | | | |
| B5 | | 1 | | | | | | | |
| B6 | 1 | | | | | | | | |
| B7 | | | | | | | 1 | | |
| B8 | | | | | | | | 1 | |
| B9 | 1 | | | | | | | | |
| B10 | | | 1 | | | | | | |
| B11 | 1 | | | | | | | | |
| B12 | 1 | | | | | | | | |
| B13 | | 1 | | | | | | | |
| B14 | 1 | | | | | | | | |
| B15 | 1 | | | | | | | 1 | |
| B16 | | | | 1 | | | | | |
| B17 | 1 | | | | | | | | |
| B18 | 1 | | | | | | | | |
| B19 | 1 | | | | | | | | |
| B20 | | | | | | | 2 | | |
| B21 | | 1 | | | | | | | |
| B22 | | | | | | | | 1 | |
| B23 | | | | | | | 1 | | |
| C1 | | | | | | | 1 | | |
| C2 | | 1 | | | | | | | |
| C3 | | | | | | | 1 | | |
| C4 | | 1 | | | | | | | |
| C5 | | | | | | | 1 | | |
| D1 | | 1 | | | | | | | |
| D2 | | | | | | | | 1 | |
| D3 | | | | | | | | 1 | |
| D4 | | | | | | | | 1 | |
| D5 | | | | | | | | 1 | |
| D6 | | | | | | | | 1 | |

| | Type of Deficiency Observed on Smoketesting Report | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Report # | Residential Lateral | City Mainline | City Storm Drain | Residential Storm | Residential Downspout | Residential Plumbing | City Manhole | Residential Cleanout | Deficiency and Number of Each Deficiency Observed on Report Page |
| D7 | 1 | | | | 1 | | | | |
| D8 | | | | | | | 1 | | |
| D9 | 1 | | | | | | | | |
| D10 | 1 | | | | | | | | |
| D11 | 1 | | | | | | | | |
| D12 | | | | | | | | | |
| F1 | 1 | | | | | | | | |
| F2 | | 1 | | | | | 1 | | |
| F3 | | | | | | | 2 | | |
| F4 | | 1 | | | | | | | |
| F5 | | 1 | | | | | | 1 | |
| F6 | | | | | | | | 1 | |
| F7 | | | | | | | | 1 | |
| F8 | | | | | | | | 1 | |
| F9 | | 1 | | | | | | | |
| F10 | 1 | | | | | | | | |
| F11 | | 1 | | | | | | | |
| F12 | | 1 | | | | | | | |
| F13 | | | | | | | | 1 | |
| F14 | | | | | | 1 | | | |
| G1 | 1 | | | | | | | | |
| G2 | | | | | | | | 1 | |
| G3 | | | | | | 1 | | | |
| G4 | | | | | | | 1 | | |
| G5 | | | | | | | | 1 | |
| G6 | 1 | | | | 1 | | | | |
| G7 | | | | | | 1 | | | |
| E1 | 1 | | | | | | | | |
| E2 | | | | | | | 1 | | |
| E3 | | 1 | | | | | | | |
| E4 | 1 | | | | | | | | |
| E5 | | 1 | | | | | | | |
| H1 | | 1 | | | | | | | |
| H2 | 1 | | | | | | | | |
| H3 | | 1 | | | | | | | |
| H4 | | 1 | | | | | | | |
| H5 | | | | | | | 2 | | |

| | Type of Deficiency Observed on Smoketesting Report | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Report # | Residential Lateral | City Mainline | City Storm Drain | Residential Storm | Residential Downspout | Residential Plumbing | City Manhole | Residential Cleanout | Deficiency and Number of Each Deficiency Observed on Report Page |
| H6 | | | | | | | 3 | | |
| I1 | | 1 | | | | | | | |
| I2 | | 1 | | | | | | | |
| I3 | | 1 | | | | | | | |
| I4 | | 2 | 1 | | | | | | |
| I5 | | | 1 | | | | | | |
| I6 | | | | | | | | 1 | |
| I7 | | | 2 | | | | | | |
| I8 | 1 | | | | 1 | | | | |
| I9 | | | | | | | 1 | | |
| I10 | 1 | | | | | | | | |
| I11 | | 1 | | | | | | | |
| I12 | | 1 | | | | | | | |
| I13 | | | 1 | | | | | | |
| I14 | | | | | | | 1 | | |
| I15 | 1 | | | | 1 | | | | |
| I16 | 1 | | | | | | | | |
| I17 | | 1 | | | | | | | |
| I18 | | | 1 | | | | | | |
| I19 | 1 | | | | | | | | |
| I20 | | 1 | | | | | | | |
| I21 | | | | 1 | | | | | |
| I22 | 1 | | | | | | | | |
| I23 | 1 | | | 1 | | | | | |
| I24 | | | | | | 1 | | | |
| I25 | 1 | | | | | | | | |
| I26 | | | 1 | | | | | | |
| I27 | | | 1 | | 1 | | | | |
| I28 | | 1 | | | | | | | |
| I29 | | | 1 | | 1 | | | | |
| I30 | | 1 | | | | | | | |
| I31 | | 1 | | | | | | | |
| I32 | 1 | | | | | | | | |
| I33 | | | | | | | | 1 | |
| I34 | | 1 | | | | | | | |
| I35 | | 1 | | | | | | | |
| I36 | | 1 | | | | | | | |

| Report # | Type of Deficiency Observed on Smoketesting Report: Residential Lateral | City Mainline | City Storm Drain | Residential Storm | Residential Downspout | Residential Plumbing | City Manhole | Residential Cleanout |
|---|---|---|---|---|---|---|---|---|
| I37 | 1 | | | | | | | |
| I38 | 1 | | | | | | | |
| I39 | 1 | | | | | | | |
| I40 | 1 | | | | | | | |
| J1 | | | | | | | 1 | |
| J2 | | | | | | | 2 | |
| K1 | 1 | | | | | | | |
| K2 | | | | | | | 1 | |
| K3 | | 1 | | | | | | |
| K4 | | | 1 | | | | | |
| K5 | | 1 | | | | | | |
| K6 | 1 | | | | | | 1 | |
| K7 | 1 | | | | | | | |
| K8 | | | | | | | 1 | |
| K9 | 1 | | | | | | | |
| K10 | | | | | | | 1 | |
| K11 | | | | | | | 1 | 1 |
| K12 | 1 | | | | | | | 1 |
| K13 | | | | | | | 1 | |
| K14 | 1 | | | | | | | |
| K15 | 1 | | | | | | | |
| K16 | 1 | | | | | | | |
| K17 | | | | | | | | 1 |
| K18 | | 1 | | | | | | |
| L1 | 1 | | | | | | | |
| L2 | 1 | | | | | | | |
| L3 | 1 | | | | | | | |
| L4 | 1 | | | | | | | |
| L5 | 1 | | | | | | | |
| L6 | 1 | | | | | | | |
| M1 | | 1 | | | | | | |
| M2 | | | | | | | | 1 |
| M3 | | | | | | | 1 | |
| N1 | | | | | | | | 1 |
| N2 | | 1 | | | | | | |
| O1 | 1 | | | | | | | |
| O2 | 1 | | | | | | | |

Deficiency and Number of Each Deficiency Observed on Report Page

| | Type of Deficiency Observed on Smoketesting Report | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Report # | Residential Lateral | City Mainline | City Storm Drain | Residential Storm | Residential Downspout | Residential Plumbing | City Manhole | Residential Cleanout |
| O3 | | | | | | | | 1 |
| O4 | | 1 | | | | | | |
| O5 | | | | | | | | 3 |
| O6 | | | 1 | | | | | |
| O7 | | | | | | | | 1 |
| O8 | | | | | | | | 1 |
| P1 | | | | | | | 1 | |
| P2 | | | | | | | | 1 |
| P3 | | | | | | | | 1 |
| P4 | | | | | | | | 1 |
| P5 | | | | | | | 1 | |
| P6 | | | | | | | 1 | |
| P7 | | | | | | | 1 | |
| P8 | | | | | | | | 1 |
| **TOTALS** | **51** | **40** | **13** | **3** | **6** | **4** | **36** | **34** |

Deficiency and Number of Each Deficiency Observed on Report Page

# APPENDIX D

APPENDIX D

**City of Toledo**

**P.O. Box 220**

**Toledo, Oregon**

**97391**

**DEPARTMENT OF ENVIROMENTAL QUALITY**

**165 E. 7TH AVE, # 100**

**Eugene Oregon**

**97401**

**NPDES PERMIT#89130** **DEC.4, 2012**

**Mr. Paul Kennedy**

**Schedule C of our discharge permit requires a sludge management plan.**

**Following is the information requested by DEQ.**

**The Toledo wastewater system consists of 113,000 feet of gravity sewers, 575 manholes, 5 pump stations and 7600 feet of force main. The treatment plant is located on 3.15 acres.**

**The average dry weather flow is .710 MGD**

**The wet weather flow is 1.7 MGD**

APPENDIX D

**The peak wet weather flows 3.5 MGD to 4.5 MGD**

**There are approximately 3560 residential, commercial, and industrial users billed for city water. This represents 90.186%residental sewage flow, 8.69%commerical sewage flow, and 1.10%industerial sewage flow. No industrial user falls under the pretreatment regulations required to discharge to the city treatment plant.**

**The city does not accept septic or chemical toilet waste. The city does provide a public RV dump station site that is located adjacent to Butler Bridge sewage lift station on Butler Bridge road. There have not been any noticeable treatment plant processing problems due to the RV holding tank dump station.**

**TREATMENT PLANT PROCESS**

**The Butler Bridge and Ammon Road lift stations pump raw influent into the new head works. Which then passes thru separate parshal flumes then combines and passes thru a hell sleeve to remove grit ¼ and larger, rags, plastics, and other undesirable materials. Which are washed compressed and discharged to a dumpster. The local sanitary company transports this material to a land fill. The raw then flows to a pista- grit removal system to remove grit ¼ in and smaller. The grit is then pumped to a grit classifier where it is washed and conveyed to a dumpster. Fecal matter and lighter material are recycled back to the flow control structure. A bar screen has been provided for a back up screening system. The raw then flows to a flow proportioning unit, then to the flow control structure where it can be directed to the aeration basin, or surge basins. At the flow control structure**

APPENDIX D

**approximately 150lbs to 200lbs of lime is added every day for alkalinity and ph control. The raw then flows to the new .191 mg aeration basin modified # 2 unit. The mixed liqueur then flows to the new .358 mg secondary clarifier. Where the settled sludge is returned to the aeration basin via a wemco pump controlled by a variable frequency drive system and floating material is removed by scum hopper and pumped to our #2 digester. The secondary effluent then flows to our chlorine contact chamber then to our two final clarifiers for contact time. Then the flow is combined in our old discharge vault where sodium bisulfate is added for decloranizion. Then the flow is measured and then discharged to the Yaquina River.**

**The plant has two treatment units the #1 unit can handle up to 1.5 MGD the #2 unit can handle up to 2.6 MGD flow. When flows exceeds 2.6 MGD the excess flow is diverted to the #2 surge basin which is the converted #2 unit clarifier where it is sent back to the flow control structure. If the event is severe enough the surge basin is isolated and becomes a primary clarifier. The primary effluent is then mixed with the completely treated effluent at the chlorine contact chamber. The #1 final clarifier has a capacity of 36,850 gallons with an average dry weather flow contact time of 156 min and 35 min peak flows. The #2 final clarifier has a capacity of 42,300 gallons with a contact time of 156 min dry weather flow and 36 min peak flow.**

**SOLIDS HANDLING**

APPENDIX D

**Waste is discharged into the #1treatment unit digester and is allowed to equalize with the #1 treatment unit aeration basin. When the unit is full we open the valve to the clarifier and allow the solids to equalize and settle. When the solids have settled in the anoxic zone we use the air lift pump and discharge them into the digester. This process is a close resemblance to the cannibal process. This also allows us to decant the supernatant back to the flow control structure. It appears that we get a reduction in solids and it thickens the solids. From there the solids are pumped to our #2 digester cell and is held under aeration until it meets the vector attraction of 38% volatile solids reduction or greater and sour of 1.5 mg/l or less. Then the bio solids are move to the holding tank for a final settling, decant, and anaerobic digestion before it is transported to the field for land application.**

**The calculations for the volatile solids are:**

$$\frac{\%\text{aeration vss} - \%\text{digester vss}}{\%\text{aeration vss} - (\%\text{aer.vss} \times \%\text{dig vss})}$$

**Calculations for specific oxygen uptake rate are as follows.**

**Oxygen uptake rate X 1000 divided by digester vss = SOUR.**

**The operating temperatures for digesters and holding tank are as follows.**

**October to May is between 12.2 and 21.1 degrees C**

**June= 20.8 degrees C**

**July= 21.2 degrees C**

APPENDIX D

**August = 20.5 degrees C**

**September = 20.1 degree C**

**PATHOGEN REDUCTION**

**Class B biosolids require less than 1,000,000 colony forming units per gram of total solids (dry weight) (expressed as geometric mean of the results of seven individual samples)**

**Seven sludge fecal samples were**

**1.318**

**2.657**

**3.502**

**4.1639**

**5.2577**

**6.6327**

**7.1033**

**Geometric mean for the seven samples = 1164 fecal count per gram of solids.**

**BIOSOLIDS PRODUCED ANNUALLY**

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**Approx. 258,000 gallons of bio solids are applied each year.**

**The average %total solids hauled is 3.32%**

**This equals 35.72 dry tons per year**

## TRANSPERTATION AND LAND APPLICATION IMPLEMENTS

**For the year 2012 the city used its new tanker to haul all 258,000 gallons of solids. The truck performed as expected.**

**The city keeps 500lbs of lime on hand at all times for any accidental spillage of bio solids either on site, during transportation, or at the application site. There are warning signs to post if required for public safety. If a digester breakdown occurs or an upset, the sludge is simply pumped to another digester until the situation is corrected.**

## BIOSOLIDS SITE MANAGEMENT INFORMATION

**Please see enclosed annual solids production forms for the following required data:**

**1. Annual biosolids production per site.**

**2. Total solids content.**

**3. Available nitrogen production**

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**4. Total pounds available nitrogen**

**5. Total acreage required to assimilate biosolids**

**6. Agronomic loading rate**

**7. Annual metal loading production**

**8. Annual metal addition/acre**

**Copies of the most recent source Biosolids analyses are included in this package of information. They will provide:**

1. **Nutrients and solids**
2. **Nitrate nitrogen**
3. **Ammonia nitrogen**
4. **Total kjeldahl nitrogen**
5. **Phosphorus**
6. **Potassium**
7. **Total solids and volatile solids**
8. **Metals**
9. **P H**

**Also enclosed are soil sample reports from all of our bio solids application sites.**

**All required information for the City of Toledo's sludge management plan has been enclosed in this report. Please review and contact me should your Dept. need any additional data.**

APPENDIX D

**Thank you**

**Gary Utiger WWTPO**

**City of Toledo**

**541-336-2138**

**E-MAIL WWTP@CITYOFTOLEDO.ORG**