# *City of Mount Vernon*
# *Comprehensive Sewer Plan Update*
# *February 2003*
# *Final*

RECEIVED
JAN 2 0 2003
DEPT OF ECOLOGY

***Prepared for:***

***City of Mount Vernon***

COPY

***Prepared by:***

***HDR Engineering, Inc.***

APPROVED
DEPARTMENT OF ECOLOGY
ENGINEERING MANAGEMENT
SIGNATURE
DATE 3/4/03

# *EXECUTIVE SUMMARY*

## INTRODUCTION

The City of Mount Vernon (Mount Vernon) has a Wastewater Utility that plans, designs, constructs, operates and maintains the City's sewerage system, pump stations, and wastewater treatment plant. The Wastewater Utility operates as the Wastewater Division of the Public Works Department.

The Mount Vernon sewerage system consists of approximately 120 miles of sewer pipe ranging in size from 6 inches to 60 inches, 1500 manholes, 11 sewage pumping stations, and a wastewater treatment plant (WWTP). The WWTP provides primary and secondary wastewater treatment utilizing the activated sludge process, with sludge stabilization by anaerobic digestion, and chlorine disinfection. The average daily flow for the year of 2001 was 3.42 MGD. The WWTP average day design flow is 5.6 MGD, with a peak design flow of 12.0 MGD. The WWTP is staffed seven days per week, and monitored during the off hours for critical system failures.

Operation, maintenance, and repair of sewerage system, pump stations and WWTP, is provided by Wastewater Division personnel. Major sewer maintenance equipment includes: two jet/vacuum trucks, video scan equipment mounted in an 18 foot van, utility pickup, and a power rodder. Public Works' Transportation Division provides additional equipment for sewer repair work that includes an excavator, backhoe, rubber tire loader, and dump trucks.

## PLANNING

The City of Mount Vernon has recently experienced the same rapid growth that is characteristic of the Puget Sound area. Sewer service is now required for many areas in the City's Urban Service Area outside those that have been studied in previous planning efforts. This growth has significant impact upon the existing and future sewer system and wastewater treatment facilities. Due to growth within the service area and continuing changes in the environmental regulations, the City has initiated planning efforts to address these issues. This has involved the completion of engineering and financial assessments to plan for the future.

The Comprehensive Sewer Plan Update - 2002 addresses the requirements of the existing combined sewer system and the developing sanitary system in order to both accommodate growth and to reduce CSOs. This is in accordance with the Revised Code of Washington (RCW) 35.67.030, which deals with sewer planning, and RCW 90.48.480, which deals with the reduction plans for combined sewer overflows. Several alternatives were evaluated in the preparation of this plan to address both of these needs. Principle concerns in the development of the plan included:

- Health and safety of the public
- Protection of the environment
- Protection of property
- Economic capability of the City

The improvements recommended in the Comprehensive Sewer Plan are consistent with the City's Comprehensive Plan. In preparing the plan, growth and CSOs were addressed together. Many of the improvements shown in the Capital Improvement Program serve both purposes.

## SEWER SYSTEM

There are two major components to the sewer system. These include the collection system and the wastewater treatment facility. The collection system includes the combined sewers in the older portions of the system with combined sewer overflows and the newer portions of the collection system which are separate sanitary sewers. Improvements required for the collection system and wastewater treatment facility were determined in the 2002 Comprehensive Plan Update and are presented in this summary.

### Combined Sewer Overflows

To protect water quality, the City is taking steps to achieve a reduction in the frequency and volume of untreated sewage discharges to the Skagit River. For several decades, the high flows during rainstorms have exceeded the capacity of the sewer and treatment facilities so the excess must be discharged to the Skagit River. These Combined Sewer Overflows are a legacy of the original sewers constructed in Mount Vernon and many other Northwest communities in the early 1900's which simply transported and dumped both sanitary sewage and storm water runoff directly into the nearest body of water.

The 1989 enlargement of the WWTP, construction of the Kulshan Interceptor in 1996, and construction of the Central CSO Interceptor in 1998 have reduced untreated overflows by more than 100,000,000 gallons annually. State and federal agencies require that significant CSO reductions be made at the earliest possible time.

The City of Mount Vernon has a consent decree with the Department of Ecology (DOE) to implement a multi-phase CSO reduction plan. Phase 1, which was completed in 1998, was construction of in-line storage. This in-line storage provided by the Central CSO Interceptor has dramatically reduced the overflows from 130 events per year down to 8. Phase 2 will add combined sewer flow capacity to the WWTP, and phase 3 (if needed) will construct a dedicated CSO treatment facility. The "Order on Consent" requires Mount Vernon to reduce overflow events to an average of one per year no later that January 1, 2015.

### Wastewater Treatment Facility Improvements:

The existing wastewater treatment facility was reviewed for future loading conditions and anticipated future effluent flows. By increasing the hydraulic capacity and making other process improvements, the plant will have the capacity to meet future flows and loadings. In addition, these improvements will reduce the number of combined sewer overflow events. These improvements include:

- Upgrade influent pump station to 24.0 million gallons per day (mgd),
- Construct a headworks, with fine screening, grit removal, disposal, and primary sludge and scum pumping facilities,
- Construct additional primary clarifiers,
- Construct an activated sludge selector basin to improve operational stability,

- Provide chemical feed system for pH control of the activated sludge system,
- Convert the Activated Sludge Pump Station to a Return Activated Sludge (RAS) Pump Station,
- Construct additional secondary clarifiers,
- Convert from chlorine to UV disinfection,
- Upgrade the capacity of the effluent pump station,
- Construct a sodium hypochlorite system for disinfecting reclaimed water for non-potable in-plant use,
- Provide Administration Building improvements, and
- Complete outfall improvements.

On a long-term planning horizon, the WWTP will need additional improvements to meet both hydraulic conveyance requirements and load requirements and anticipated treatment requirements. Anticipated future discharge permit requirements may necessitate modifications to the process to provide nitrification. Improvements to convert the activated sludge system to a nitrification mode, hydraulic improvements, and other process or site improvements are listed as follows:

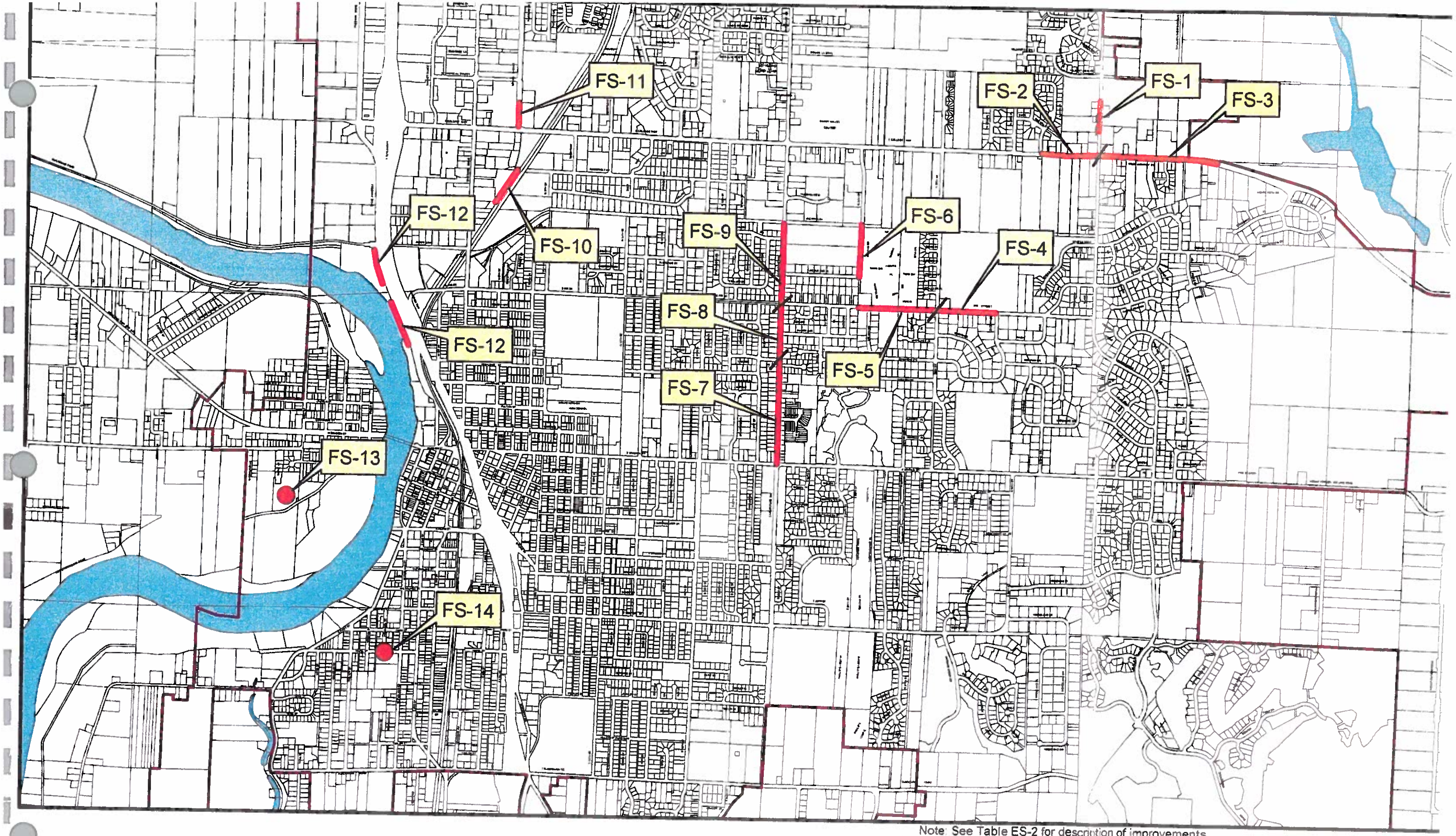
- Convert the Aeration Basin No. 4 to an Activated Sludge Aeration Basin,
- Construct additional Aeration Basins,
- Convert Secondary Clarifier No. 1 to an Aerobic Digester,
- Construct an additional Secondary Clarifier,
- Construct an additional Dissolved Air Floatation Thickener,
- Construct an additional Anaerobic Digester,
- Provide expansion to the existing laboratory,
- Provide odor control, and
- Acquire adjacent land for possible ring dike construction for flood protection, and as an odor and noise buffer.

The proposed wastewater treatment plant improvements are shown on Figure ES-1 and costs for these are presented in Table ES-1, included at the end of this summary.

**Sewer System Improvements**

The Mount Vernon sewer system totals approximately 120 miles of sewer pipe. Portions of the system were constructed in the early 1900's, and much of the system is 60 years or older. As a part of the Comprehensive Sewer Planning study, the interceptor conveyance system was evaluated to determine improvements that would be required for additional capacity for future growth within the existing service area. These are identified as improvements FS-1 through FS-14, summarized in Table ES-2 and shown on Figure ES-2 included at the end of this summary.

Existing City information was reviewed to determine areas where repair and replacement is recommended. This includes areas within the older combined portion of the sewer system and the typical type of defects identified included structural damage and areas where root intrusion has occurred. These are summarized in Table ES-3 along with estimated repair and replacement costs.




Note: See Table ES-2 for description of improvements

N

0 1,000 2,000 3,000 4,000 Feet

| | |
|---|---|
| Mount Vernon<br>Comprehensive Sewer Plan | Date:<br>Dec 2002 |
| Interceptor Improvements | Figure No.<br>FS-1 |

Improvements for future development were determined from a model of the major interceptors of the City. Improvements required for ultimate build-out of the City are identified in the Sewer Comprehensive Plan. These improvements will be required in the future, and timing of the improvements is dependant upon actual growth patterns within the City.

**Sewer Service to Areas within Urban Growth Areas**

A number of improvements will be required to extend sewer service into the UGA and other developing areas. These are areas within the UGA, but not currently within the City limits. The City is presently initiating a study to determine the necessary improvements needed within each of these four areas to provide sewer service. It is the City's intent to determine the services that will ultimately be required, and then develop a phased approach that can be implemented as the need occurs. This will provide an overall cost effective system from both a capital and operating standpoint.

To allow interim development within the UGA areas that are currently without sewer, the City has adopted sewer development provisions in the City of Mount Vernon Sewer Ordinance Title 13. These provisions allow limited interim commercial and industrial development by permitting use of onsite storage systems, and allow limited residential development with onsite septic systems.

South Mount Vernon

The Mount Vernon Overall Economic Development Plan lists South Mount Vernon Planning as the number one implementation plan priority project. The 1996, Overall Economic Development Plan (OEDP) schedule for implementing the South Mount Vernon plan is 3-6 years. Sewer construction was completed for commercial areas adjacent to Old Highway 99 from Blackburn Road to Hickox Road in 2002. Sewer construction is scheduled for the commercial area adjacent to Cedardale Road from Anderson Road to Hickox Road in 2003. The extension of sewers to residential areas within the South UGA will be developer or LID funded. Full build-out of the UGA will require improvements to sewer interceptors within the City boundary.

West Mount Vernon

The Plan assumes that areas to the west of Mount Vernon will remain primarily agricultural. The City has reviewed the development potential in West Mount Vernon along Memorial Highway to the UGA boundary. Based on preliminary review it appears that serving this area will require construction of 6,000 feet of gravity sewer and at least one pumping station with 3,000 feet of force main. There may be some opportunity for phasing development; however, the first phase would require construction of the pump station and force main. The collection sewers into the West Mount Vernon pump station and the pump station itself would also need to be evaluated to determine if additional improvements are required. The extension of sewers to residential and commercial areas within the West UGA will be developer or LID funded. Full build-out of the UGA will require improvements to sewer Interceptors within the City boundary.

North Mount Vernon

Sewer capacity on Francis Road was improved in 2002 and is adequate for projected design flows in the Northern UGA. Sewer alignments and pump station locations for the Northern UGA have not been determined. The extension of sewers to the Northern

UGA will be developer or LID funded. Full build-out of the UGA will require improvements to sewer interceptors within the City boundary.

East Mount Vernon

A significant portion of the Eastern UGA is tributary to the Big Lake Sewer System (Skagit Public Utility District No. 2). The City of Mount Vernon will coordinate with the PUD No. 2, and other stakeholders to identify and implement an efficient sewer service plan. The Comprehensive Sewer Plan proposes extending sewer along College Way to Highway 9, and South along Highway 9 to Division Street. Development of the Eastern UGA will require construction of regional pumping facilities. Pump stations that do not provide regional service will not be allowed. Sewer alignments and pump station locations for the Eastern UGA have not been determined. The extension of sewers to residential and commercial areas of the Eastern UGA will be developer or LID funded. Full build-out of the UGA will require improvements to sewer interceptors within the City boundary.

## SEWER UTILITY FUNDING

The City adopted a sewer rate ordinance for the years 2000 - 2004. The rate plan covers operation, maintenance, debt payment and debt coverage based on year 2000 projections.

Other funding sources include developer charges for sewer expansion and sewer repair/replacement. The Wastewater Utility is planning a review of service rates and developer charges prior to expiration of the current rate ordinance.

## LEVEL OF SERVICE STATEMENT

It is the goal of the City to minimize degradation of water quality and to maintain compliance with the requirements of the City's Washington Department of Ecology Wastewater Discharge Permit. An ongoing program of sewer system repair and replacement, and enforcement of development standards, will contribute to the reduction of combined sewer overflows, sewer system infiltration and exfiltration. These efforts will promote health and safety of the public, protection of the environment, and enhance the economic vitality of the City.

## CAPITAL IMPROVEMENT COSTS

Capital improvement program costs for the period from the year 2001 through 2020 are summarized in Table ES-4.

## SEPA COMPLIANCE

The City of Mount Vernon has received a SEPA Determination of Non-Significance (DNS) for the Comprehensive Plant Upgrade in November 2000. A copy of the DNS is included in Appendix O.

## TABLE ES-1

| Recommended Improvements for the Wastewater Treatment Plant | |
|---|---|
| **Improvement** | **Capital Cost Estimate (1,000)** |
| Influent Pump Station | $1,6000 |
| Headworks | $2,800 |
| Primary Clarifiers | $1,800 |
| Selector Basins | $600 |
| Aeration Basins | $2,700 |
| Chemical Feed System (pH control) | $50 |
| Secondary Clarifiers | $3,600 |
| UV Disinfection[2] | $1,340 |
| Effluent Pump Station | $370 |
| Outfall | $1,200 |
| Sodium Hypochlorite Sytem | $100 |
| DAFT | $400 |
| Anaerobic Digester | $2,500 |
| Odor Control System | $1,300 |
| Administration Building | $500 |
| Laboratory Expansion/Operations Center | $600 |
| Shop and Garage | $500 |
| Flood Protection – 100 year event | $600 |
| Roadways | $250 |
| Drainage Improvements | $50 |
| TOTAL | $23,593 |

1. ENR Construction Cost Index 6397, October 2001.
2. UV disinfection costs include capital cost of a UV disinfection system and costs for pilot testing for two (2) months.

## Table ES-2

| Interceptor System Improvements | | | | | | |
|---|---|---|---|---|---|---|
| ID No. | Location | Between | Year Required | Dia (in)[1] | Length (ft)[1] | Cost ($1,000)[2] |
| FS-1 | Martin Road | Trumpter Rd. and College Way | As required | 12 | 734 | 135 |
| FS-2 | College Way | Martin Rd and 35th Street | As required | 15 | 548 | 125 |
| FS-3 | College Way | Martin Rd to Pump Station | 2002 | 18 | 2,307 | 635 |
| FS-4 | Fir Street | 30th Str. and Comanche Drive | 2005 | 18 | 980 | 270 |
| FS-5 | Fir Street | 30th Str. and 26th Street | 2005 | 18 | 1,265 | 350 |
| FS-6 | 26th Street | Jacqueline Place and Kulshan Avenue | As required | 18 | 690 | 190 |
| FS-7 | LaVenture Road | Division Str. and Cascade Street | As required | 10 | 1,525 | 235 |
| FS-8 | LaVenture Road | Cascade Str. and Fir Street | As required | 10 | 495 | 75 |
| FS-9 | LaVenture Road | Fir Str. and Kushan Avenue | As required | 12 | 1,386 | 255 |
| FS-10 | Alder Lane Interceptor | Burlington Northern Railroad of Roosevelt Avenue | As required | 24 | 600 | 220 |
| FS-11 | Urban Avenue | North of College Way | As required | 12 | 375 | 70 |
| FS-12 | Freeway Drive | River Bend Road and Cameron Way | As required | 12 | 1,309 | 240 |
| FS-13 | West Mount Vernon | Modify Pump Station | As required | | | 150 |
| FS-14 | Central CSO Regulator | Add Fail-Safe Gate Operator | 2001 | | | 30 |

1. Improvements are based on saturated development, based on the UGA boundary, 100 gpcd, 1, 100 gpad (inflow and infiltration), and L.A. Peaking curve.
2. Costs are based on ENR Cost index of 6390 (October 2001), and include restoration, 25% for legal, administration, and engineering costs, 7.8% for sales tax, and a 20% contingency.

## TABLE ES-3

| Collection System Improvements | | | | | |
|---|---|---|---|---|---|
| ID No. | Location | Defect | Defect identified Via | Improvement | Cost ($1,000)[1] |
| CS-1 | Snoqualmie, MH B29A to MH B29 | Root intrusion | Video[2] | Remove roots and Slipline with 300 LB | $20 |
| CS-2 | Yard of house 1115 N0. 8th, MH 49 to MH 50 | Root intrusion | Video[2] | Remove roots and Slipline with 250 LB | $20 |
| CS-3 | So. 7th and Jefferson to So. 7th and Washington, MH 39 to MH 37 | Root intrusion | Video[2] | Remove roots and Slipline with 450 LB | $20 |
| CS-4 | No. 6th and Lawrence, MH C39 to MH C38 | Root intrusion | Video[2] | Remove roots and Slipline with 320 LB | $20 |
| CS-5 | Brick Hill, MH 01, North along I-5 | Root intrusion | Video[2] | Remove roots and Slipline with 400 LB | $20 |
| CS-6 | Blodgett Rd to North of Blackbur, MH 55 to MH 54 | Root intrusion | Video[2] | Remove roots and Slipline with 270 LB | $20 |
| CS-7 | Kincaid, MH 25, to MH 23 | Root intrusion | Video[2] | Remove roots and Slipline with 240 LB | $20 |
| CS-8 | So. 20th, North off Section, MH 32 to MH 31 | Root intrusion | Video[2] | Remove roots and Slipline with 120 LB | $20 |
| CS-9 | Section, MH D33 to between MH D32-D31 | Structural Damage | Video[2] | Replace with 420 LF of 8-inch pipe | $50 |
| CS-10 | Alley between Douglas and Walter, MH A13 to A05 | Structural Damage | Video[2] | Replace with 640 LF of 8-inch pipe | $75 |
| CS-11 | 107 Cedar to the South, MH F11 to F29 | Structural Damage | Video[2] | Replace with 300 LF of 8-inch pipe | $45 |
| CS-12 | No. 6th, MHF13 to F14 | Structural Damage | Video[2] | Replace with 400 LF of 8 | $60 |
| CS-13 | Section and Rail Road Ave, MH E17 to E18 | Structural Damage | Video[2] | Sport repair-verify grease problem is corrected | $5 |

**Table ES-3 cont.**

| Collection System Improvements | | | | | |
|---|---|---|---|---|---|
| **ID No.** | **Location** | **Defect** | **Defect identified Via** | **Improvement** | **Cost ($1,000)[1]** |
| CS-14 | Broadway at alley between So. 9th & 10th, MH D41 to D40 | Structural Damage | Video[2] | Slipline with 330 LF | $20 |
| CS-15 | Broad, east of So. 11th, MH 54 to MH 49 | Structural Damage | Video[2] | Replace with 230 LF of 8-inch pipe | $20 |
| CS-16 | Line under I-5 | Structural Damage | Video[2] | Will require further | --[4] |
| CS-17 | Alley, north of Division, east of No. 11th, MH C66 to C65 | Structural Damage | Video[2] | Spot Repair | $5 |
| CS-18 | Bernice, east of So. 14th, MH G42 to G41 | Structural Damage | Video[2] | Spot Repair | $5 |
| CS-19 | So. 3rd and Vera, MH A41 to I42 | Structural Damage | Video[2] | Pipe has been | -- |
| CS-20 | Lawrence and 7th, MH C73 | Structural Damage | Video[2] | Spot Repair | $5 |
| CS-21 | 1224 12th Str. So, between MH G8 and G11 | Structural Damage | Video[2] | Replace with 200 LF of 8-inch pipe | $25 |
| CS-22 | 117th North 8th Str. | Flooding | Data Base[3] | See 8th Str. Section[3] | --[5] |
| CS-23 | 420 E. Fulton | Flooding | Data Base[3] | See 8th Str. Section[3] | --[5] |
| CS-24 | 919 W. Division | Flooding | Data Base[3] | No improvements-surface flooding problem | -- |
| CS-25 | Alley at Carpenter, between So 9th and so. 10th heading north to Division | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-26 | 1120 No 16th, 340 ft north of MH M68 on Florence and 16th | Cracked Pipe | Data Base[3] | Spot Repair | $5 |

**Table ES-3 cont.**

| ID No. | Location | Defect | Defect Identified Via | Improvement | Cost ($1,000)[1] |
|---|---|---|---|---|---|
| CS-27 | 1210 N. 14th, north of Florence and 14th | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-28 | 8th Str. And Evergreen heading north, F18 to F15 | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-29 | 7th and Warren, toward Fulton, MH C73 to C72 | Cracked Pipe | Data Base[3] | See 8th Str. Section | --[5] |
| CS-30 | 16th and Blackburn heading east 17th, J08 to J09 | Obstruction | Data Base[3] | Jet main and monitor flows | -- |
| CS-31 | 100 Washington-storm line going to SE under I-5, MH C19 to C20 | Cracked Pipe | Data Base[3] | Will require further assessment | --[4] |
| CS-32 | Scott's Bookstore, N 1st to N 1st and Division | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-33 | Snoqualmie St. between Cleveland and S 2nd Str. MH B32 to B03 | Cracked Pipe | Data Base[3] | Reassess slipline if necessary | -- |
| CS-34 | Westside of Christenson Seed West to So 3rd, MH E01 to A39 | Infiltration | Data Base[3] | Spot Repair | $5 |
| CS-35 | Cleveland and Blackburn to just West of Harrison and Blackbum, MH J11 to J09 | Infiltration, Joint problem | Data Base[3] | Slipline 300 LF | $20 |
| CS-36 | N Laventure just south of E Fir to N Laventure just north of E Fir, MH N06 to N04 | Root intrusion | Data Base[3] | Reassess slipline if necessary | -- |
| CS-37 | North of Casecade Str., on N Laventure to S of E Fir on Laventure, MH N08 to N06 | Root intrusion | Data Base[3] | Reassess slipline if necessary | -- |

**Table ES-3 cont.**

| ID No. | Location | Defect | Defect identified Via | Improvement | Cost ($1,000)[1] |
|---|---|---|---|---|---|
| CS-38 | N Laventure, Fulton to Cascade, MH N12 to N10 | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-39 | Hoag Rd., Parkway Dr., to Hoag Rd | Root intrusion | Data Base[3] | Reassess slipline if necessary | -- |
| CS-40 | Lind Str. And S. 6th to N on S 6th, MH E76 to E75 | Infiltration | Data Base[3] | Spot Repair | $5 |

[1] Costs are based on ENR Cost index of 6390 (October 2001), and include restoration, 25% for legal, administration, and engineering costs, 7.8% for sales tax, and a 20% contingency.
[2] Defect identified via review of video records.
[3] Defect identified via review of City Sewer Data Base.
[4] Interstate-5 Crossings are estimated at $750,000 for all nine improvements.
[5] 8th Street improvements have been estimated at $1,000,000 to correct the localized surcharging.

## Table ES-4

| Capital Improvement Program Cost ($1,000)[1] | | | | |
|---|---|---|---|---|
| Year(s) | Wastewater Conveyance System[1] | Wastewater Treatment Facility[1] | Combined Sewer System Treatment[2] | Total[1] |
| 2001 | $570 | $0 | $0 | $570 |
| 2002 | $635 | $350 | $0 | $985 |
| 2003 | $1,000 | $1,200 | $0 | $2,200 |
| 2004 | $750 | $11,940 | $0 | $12,690 |
| 2005 | $620 | $0 | $0 | $620 |
| 2006 | --[3] | $0 | $0 | $0 |
| 2011-2020 | $2,510 | $9,800 | $9,100 | $21,410 |
| TOTAL | $6,085 | $23,290 | $9,100 | $38,475 |

1. ENR Construction Cost Index 6397, October 2001.
2. Detailed costs are provided in Chapter 5 and Chapter 10.
3. Improvements during these years are expected to be identified as necessitaty dictates, and costs are included in the future cost estimates.

## TABLE OF CONTENTS

1. INTRODUCTION ..... 1

2. SYSTEM DESCRIPTION ..... 2

3. BASIC PLANNING DATA ..... 5

4. COMBINED SEWER SYSTEM ..... 24

5. WASTEWATER COLLECTION SYSTEM ..... 43

6. INDUSTRIAL PRETREATMENT ..... 68

7. EXISTING WASTEWATER TREATMENT PLANT ..... 81

8. WASTEWATER TREATMENT PLANT ANALYSIS ..... 91

9. WASTEWATER TREATMENT PLANT ALTERNATIVES ..... 108

10 RECOMMENDED WWTP ALTERNATIVES ..... 147

11. CAPITAL IMPROVEMENT PLAN ..... 156

## LIST OF TABLES

Table 2-1 City of Mount Vernon's Sanitary Sewer System Pump Stations ..... 3

Table 3-1 City of Mount Vernon Population Projections and Service Area Population Projections ..... 9

Table 3-2 Combined Sewer Component Flow and Load Projections for 2020 ..... 14

Table 3-3 Historical Flows for the City of Mount Vernon ..... 18

Table 3-4 Historical Average Month Load for the City of Mount Vernon ..... 18

Table 3-5 Flow Projections for the City of Mount Vernon ..... 20

Table 3-6 Projected BOD Loadings for the City of Mount Vernon ..... 21

Table 3-7 Projected TSS Loadings for the City of Mount Vernon ..... 21

Table 3-8 Projected $NH_4$-N Loadings for the City of Mount Vernon[1] ..... 22

Table 3-9 WWTP and CSO Flow and Load Projections ..... 23

Table 4-1 City of Mount Vernon's Combined Sewer Overflow Pump Stations ..... 25

Table 4-2 Combined Sewer Overflows from November 1998 to 2000 ..... 26

Table 4-3 Summary of CSO Treatment Alternatives ..... 34

Table 4-4 Summary of CSO Reduction Plan Improvements ..... 40

Table 4-5 Recommended Improvements for the CSO Treatment Facility ..... 42

Table 5-1 Hydraulic Analysis Identified Capacity Limitations at Saturated Development ..... 47

Table 5-2 Interceptor System Improvements ..... 48

Table 5-3 Interstate 5 Crossings ..... 56

Table 5-4 Collection System Improvements ..... 61

Table 5-5 Repair and Replacement Program ..... 67

Table 6-1 Historical Flows and Loads for Draper Valley Farms, Inc. ..... 71

Table 7-1 Dissolved Oxygen Total Maximum Daily Load for Mount Vernon for the Skagit River ..... 82

Table 7-2 NPDES Permit Effluent Limits for Conventional Pollutants for the Mount Vernon WWTP ....83

Table 7-3 NPDES Permit Effluent Limits for Chemical Pollutants for the Mount Vernon WWTP ....83

Table 8-1 Skagit River BOD and $NH_3$ TMDL Limits ....91

Table 8-2 Estimated $BOD_5$ and $NH_3$ Loadings for the Skagit River during the Time Average Monthly TMDL Limits Apply (July – October) ....92

Table 8-3 2020 Process Capacity Analysis with ENVision Model ....96

Table 8-4 Summary of Requirements to Meet 2010 Flows and Loads ....106

Table 8-5 Summary of Requirements to Meet 2020 Flows and Loads ....107

Table 9-1 Influent Pump Station: Alternative A Cost Estimate (Upgrading Existing Wetwell/Drywell Pump Station) ....111

Table 9-2 Influent Pump Station: Alternative B Cost Estimate (Covert to Submersible Pump Station) ....112

Table 9-3 Evaluation of Grit Removal Alternatives ....118

Table 9-4 Capital Costs ($1,000) for 25.8 mgd Primary Clarifier Alternatives ....121

Table 9-5 Aeration Basin Improvements Estimated Project Cost ....126

Table 9-6 Cost for Secondary Clarifiers ....128

Table 9-7 Life Cycle Costs (in $1,000) for 25.8 mgd Disinfection Alternatives ....131

Table 9-8 Single Pipe Outfall (Alternative A) Cost Estimates ....136

Table 9-9 Two Pipe Outfall (Alternative B) Cost Estimates ....136

Table 9-10 Outfall Alternative Advantages and Disadvantages ....137

Table 9-11 Co-generation with Microturbines Costs Estimates ....139

Table 9-12 Odor Control Cost Estimate ....140

Table 9-13 Water Quality Classifications for Reclamation End-Uses ....144

Table 9-14 Estimated Capital Cost of 1 MGD Reclaimed Water Treatment System and Distribution Infrastructures ....145

Table 10-1 Recommended Improvements for the Wastewater Treatment Plant ....153

Table 11-1 WWTP Capital Improvement Schedule 2000-2020 ($1,000) .....156

Table 11-2 CSO Treatment Improvement Schedule 2000-2020 ($1,000).....160

Table 11-3 Collection System Improvement Schedule 2000-2020 ($1,000) .....161

Table 11-4 Summary of Capital Improvement Schedule 2000-2020 ($1,000).....165

## LIST OF FIGURES

Figure ES-1 Recommended WWTP Improvements Site Plane ..... 4

Figure ES-2 Interceptor Improvements..... 5

Figure 3-1 Existing Collection System ..... 7

Figure 3-2 City of Mount Vernon Urban Growth Area ..... 8

Figure 3-3 Mount Vernon Daily WWTP Flows and Rainfall, July 1 – December 31, 1999 .....10

Figure 3-4 City of Mount Vernon Monthly WWTP Flows.....10

Figure 3-5 Idealized Combined Sewer Flow Hydrograph, May 16, 1988 .....13

Figure 3-6 City of Mount Vernon Monthly BOD Loadings .....15

Figure 3-7 City of Mount Vernon Monthly TSS Loading.....16

Figure 3-8 City of Mount Vernon Ammonia Nitrogen Influent Concentration.....17

Figure 3-9 City of Mount Vernon Monthly Ammonia Loading.....17

Figure 4-1 City of Mount Vernon Combined Sewer System Flows, Cumulative Flows for December 29, 1998 .....27

Figure 4-2 City of Mount Vernon Monthly Flow vs. Rainfall .....28

Figure 4-3 Alternative 1 CSO Treatment Facility Schematic.....31

Figure 4-4 Alternative 2 CSO Treatment Internal Shunt Schematic.....32

Figure 4-5 Alternative 3 CSO Treatment Internal Shunt Schematic.....33

Figure 4-6 Recommended Process Schematic Flow Diagram.....36

Figure 5-1 Collection System – Conveyance, Pump Stations, and Overflow Structures .....44

Figure 5-2 Drainage Area Basins .....46

Figure 5-3 Future Interceptors.....50

Figure 5-4 North 8th Street Improvements .....58

Figure 5-5 Interstate 5 Sewer Crossings .....59

Figure 6-1 DVF Wastewater Discharges – BOD (lbs. per day) .....72

Figure 6-2 DVF Wastewater Discharges – suspended Solids (lbs. per day) .....73

Figure 7-1 Existing Hydraulic Profile.....85

Figure 9-1 Alternate WWTP Hydraulic Profile.....109

Figure 9-2 Influent Pump Station Upgrade Alternative A – Plan .....113

Figure 9-3 Influent Pump Station Upgrade Alternative A – Typical Section .....114

Figure 9-4 Influent Pump Station Upgrade Alternative B – Plan .....115

Figure 9-5 Influent Pump Station Upgrade Alternative B – Typical Section.....116

Figure 9-6 Proposed Headworks Facility.....119

Figure 9-7 Proposed Primary Clarifiers .....122

Figure 9-8 Proposed Aeration Basins and Secondary Clarifiers .....127

Figure 9-9 Proposed UV Disinfection and Effluent Pump Station .....134

Figure 9-10 Conceptual Reclaimed Water Forcemain Alignment .....146

Figure 10-1 Site Plan of Recommended Improvements .....154

Figure 10-2 Site Plan of Recommended Yard Piping.....155

## APPENDICES

Appendix A – Draper Valley Farms, Inc. 20-Year Flow Projections

Appendix B – L.A. Peaking Curve

Appendix C – The City of Mount Vernon's Basin Delineation for Hydraulic Modeling

Appendix D – Hydraulic Analysis Output of the Clty of Mount Vernon's Wastewater Collection System

Appendix E – Draper Valley Farms, Inc. Draft Industrial Pretreatment Report Comments

Appendix F – Meeting Minutes from January 9, 2001, Meeting between City of Mount Vernon Staff, Department of Ecology Representatives, and HDR Engineering

Appendix G – National Pollutant Discharge Elimination System Permit for the City of Mount Vernon

Appendix H – ENVision Model Data Summary sheets for the City of Mount Vernon Wastewater Treatment Plant

Appendix I – City of Mount Vernon WWTP Outfall Permits and Schedule Assessment

Appendix J – Mount Vernon WWTP UV Transmittance Test Results

Appendix K – Mount Vernon WWTP Mixing Zone Study

Appendix L – WaterWorld™ Article on Microturbines

Appendix M – Technical Mernorandum Aeration Basin Upgrade

Appendix N – Staffing Calculations

Appendix 0 – Determination of Non-Significance (DNS)

# ABBREVIATIONS

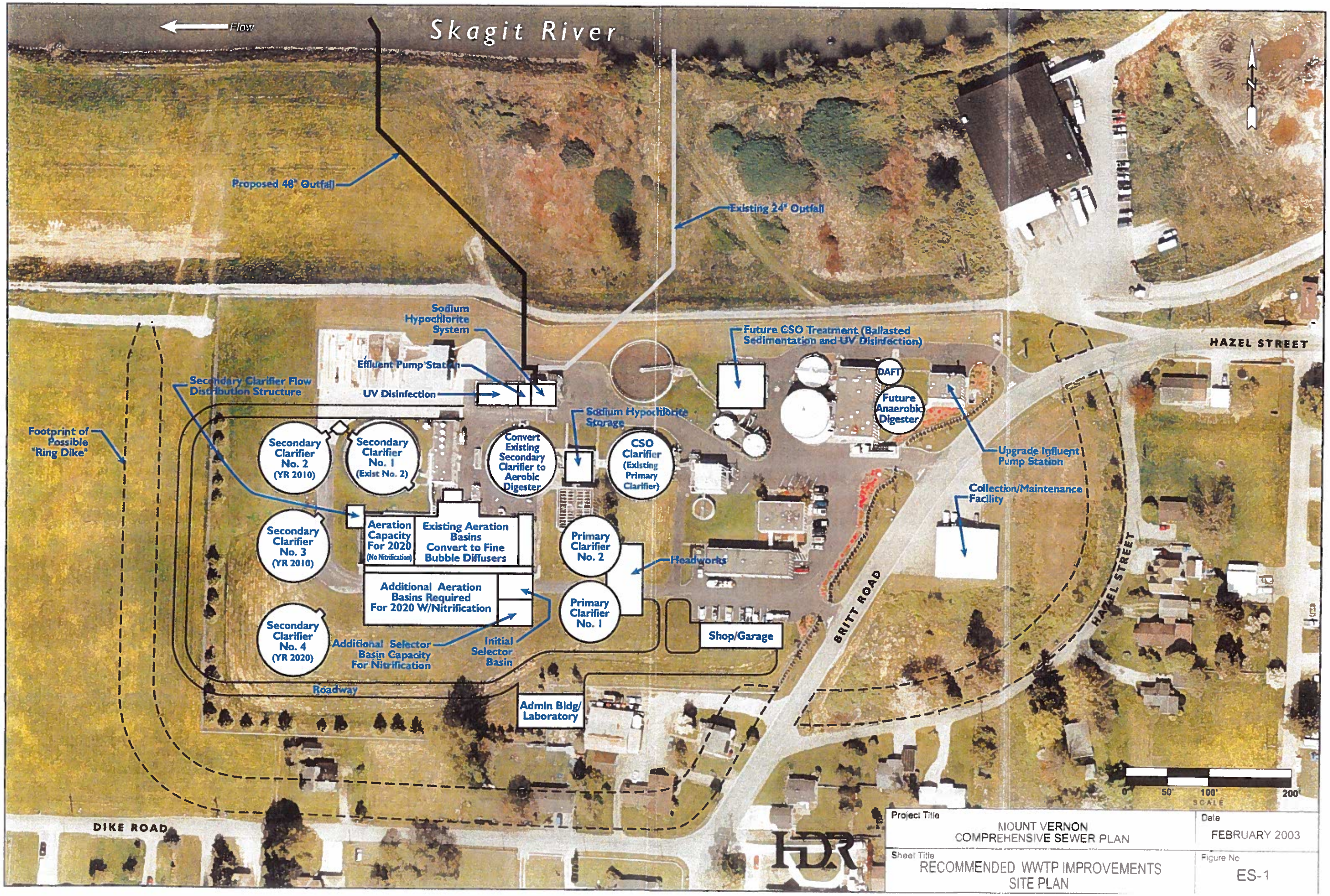
| | |
|---|---|
| ac | Acre |
| ADMM | Average Day Maximum Month |
| AKART | All known, available, and reasonable methods of treatment |
| BAT | Best Available Technology Economically Achievable |
| BE/BA | Biological Evaluation/Biological Assessment |
| BOD | Biochemical Oxygen Demand |
| BPT | Best Practical Control Technology Currently Available |
| $CBOD_5$ | Carbonaceous 5-day biochemical oxygen demand |
| cf | Cubic feet |
| cfs | Cubic feet per second |
| cfu | Colony forming units |
| cfu/100 mL | Colony forming units per 100 milliliters |
| CSO | Combined Sewer Overflow |
| DAF | Dissolved air flotation |
| DAFT | Dissolved Air Floatation Thickener |
| DO | Dissolved oxygen |
| DOE | Department of Ecology |
| DVF | Draper Valley Farms, Inc. |
| EPA | Environmental Protection Agency |
| FEB | Flow equalization basin |
| fps | Feet per second |
| ft/mg | Feet per million gallons |
| GMA | Growth Management Act |
| gpad | Gallons per acre per day |
| gpcd | Gallons per capita per day |

# ABBREVIATIONS

| | |
|---|---|
| gpd | Gallons per day |
| gpd/sf | Gallons per day per square foot |
| gpm | Gallons per minute |
| HGL | Hydraulic grade line |
| hp | Horsepower |
| HRT | Hydraulic residence time |
| kcf | 1,000 cubic feet |
| kW | Kilowatt |
| KWhr | Kilowatt-hour |
| lb/day | Pounds per day |
| lb/d-sf | Pounds per day per square foot |
| lb/hr-sf | Pounds per hour per square foot |
| LF | Linear foot |
| LS | Lump sum |
| mg | Million gallons |
| mg/L | Milligrams per liter |
| mgd | Million gallons per day |
| mL/L/hr | Milliliters per liter per hour |
| MLSS | Mixed Liquor Suspended Solids |
| $NH_3$ | Ammonia |
| NPDES | National Pollutant Discharge Elimination System |
| OFR | Overflow Rate |
| ppcd | Pounds per capita per day |
| ppd | Pounds per day |
| psi | Pounds per square inch |
| RAS | Return activated sludge |

# ABBREVIATIONS

| | |
|---|---|
| scfm | Standard cubic foot per minute |
| sf | Square foot |
| SRT | Solids residence time |
| TMDL | Total maximum daily loads |
| TSS | Total Suspended Solids |
| UGA | Urban Growth Area |
| UV | Ultraviolet disinfection |
| VSS/kcf-d | Volatile suspended solids per 1,000 cubic feet per day |
| WAC | Washington Administrative Code |
| WAS | Waste Activated Sludge |
| WDNR | Washington Department of Natural Resources |
| WDOE | Washington Department of Ecology |
| WLA | Waste Load Allocations |
| WSEL | Water surface elevation |
| WWTP | Wastewater Treatment Plant |


Flow
Skagit River
Proposed 48" Outfall
Existing 24" Outfall
Sodium Hypochlorite System
Effluent Pump Station
UV Disinfection
Secondary Clarifier Flow Distribution Structure
Footprint of Possible "Ring Dike"
Secondary Clarifier No. 2 (YR 2010)
Secondary Clarifier No. 1 (Exist No. 2)
Convert Existing Secondary Clarifier to Aerobic Digester
Sodium Hypochlorite Storage
CSO Clarifier (Existing Primary Clarifier)
Future CSO Treatment (Ballasted Sedimentation and UV Disinfection)
DAFT
Future Anaerobic Digester
Upgrade Influent Pump Station
Collection/Maintenance Facility
HAZEL STREET
Secondary Clarifier No. 3 (YR 2010)
Aeration Capacity For 2020 (No Nitrification)
Existing Aeration Basins Convert to Fine Bubble Diffusers
Primary Clarifier No. 2
Headworks
Additional Aeration Basins Required For 2020 W/Nitrification
Primary Clarifier No. 1
Secondary Clarifier No. 4 (YR 2020)
Additional Selector Basin Capacity For Nitrification
Initial Selector Basin
Shop/Garage
BRITT ROAD
Roadway
Admin Bldg/ Laboratory
DIKE ROAD
0 50' 100' 200'
SCALE
HDR
Project Title
MOUNT VERNON COMPREHENSIVE SEWER PLAN
Date
FEBRUARY 2003
Sheet Title
RECOMMENDED WWTP IMPROVEMENTS SITE PLAN
Figure No
ES-1

# 1. INTRODUCTION

## AUTHORIZATION

In May of 2000, the City of Mount Vernon authorized HDR Engineering to proceed with updating the City's Comprehensive Sewer Plan.

## PURPOSE

The purpose of this update was to investigate and review the existing wastewater conveyance system and wastewater treatment facility. This included a review of the system operation and development of an improvement plan to meet future system needs. The development of this plan included:

- Reviewing existing flows and loads and estimating future flows and loads.
- Assessing the capability of the existing conveyance system and wastewater treatment plant to meet existing and future flows and loads.
- Develop the least costly system improvements to meet existing and future requirements.

The results of these investigations are presented in this report as a plan for expansion, operation, and maintenance of the wastewater conveyance system and wastewater treatment facility to comply with the requirements of the Washington State Department of Ecology as set forth in their rules and regulations, WAC 173-240 and WAC 173-245.

## ACKNOWLEDGEMENTS

The suggestions, contributions, and assistance provided by the City's staff were invaluable in the preparation of this report.

# 2. SYSTEM DESCRIPTION

## SYSTEM BACKGROUND

Mount Vernon, Washington, is situated approximately half way between Seattle and the Canadian Border. It ranks first in size among the major communities in Skagit County.

Potable water supply to Mount Vernon is provided by the Skagit County Public Utility District (PUD) No. 1, the eleventh largest water provider in the State of Washington. Water diverted from the Cultus Mountain streams is stored in the recently upgraded 1.45 billion gallon Judy Reservoir. After Treatment at the Judy Reservoir Water Treatment Plant, finished potable water is supplied to Mount Vernon via the existing transmission pipeline. At present, the Skagit County PUD No. 1 is constructing a Skagit River Pump Facility to provide an alternate raw water supply to the Judy Reservoir, expanding the treatment capacity of the water treatment plant, and constructing of a new transmission line to Mount Vernon.

At present, the maximum pumping capacity to Mount Vernon is 18 million gallons per day. The annual average consumption is estimated to be 7 million gallons per day; the annual peak consumption is 14 million gallons. Basic charge is $11.40 per month per single family dwelling. From 0 to 600 cubic feet the charge is $1.43 per c.f.; over 600 cubic feet the charge is $1.93 per c.f. There is a $10 connection fee, and first-time users are required lo make a $100.00 refundable deposit.

The City of Mount Vernon provides the wastewater services and the following sections provide a summary description of the existing system.

## OWNERSHIP AND MANAGEMENT

The City of Mount Vernon provides treatment and conveyance of domestic, industrial, and commercial wastewaters within the City's UGA. The one large industrial customer currently served is Draper Valley Farms, Inc. which is a chicken processing facility.

## EXISTING FACILITIES INVENTORY

### Summary

The existing sewer system consists of both sanitary and combined sewers. The combined sewers are limited to the older portions of the City. Gravity sewers range in size from 6-inch to 60-inch pipes. Combined service area is approximately 2 square miles and the separated service area covers approximately 14 square miles. The total service area is served by approximately 120 miles of pipe. A majority of the pipe materials are concrete, but clay, corrugated metal, and PVC have also been utilized. Major interceptors, pump

stations, combined sewer overflow structures, and the wastewater treatment plant are identified below.

### Interceptors

The major interceptors in the City are:

- Central Interceptor;
- West Interceptor;
- Kulshan Interceptor;
- Alder Lane Interceptor; and
- Southeast Interceptor.

These convey all flows to the wastewater treatment plant.

### Pump Stations

Mount Vernon's wastewater flows are conveyed to the treatment plant through a series of pump stations. The conveyance system pump stations are presented in Table 2-1.

**Table 2-1**

| City of Mount Vernon's Sanitary Sewer System Pump Stations | | | |
|---|---|---|---|
| **Pump Station** | **Type** | **No. of Pumps** | **Firm Pumping Capacity (gpm)** |
| Alder Lane | Submersible | 4 | 2,800 |
| East College Way | Submersible | 2 | 380 |
| Hoag Road | Submersible | 2 | 200 |
| Martin Road | Submersible | 2 | 200 |
| Freeway Drive | Submersible | 2 | 350 |
| Maple Way | Wet well/dry well | 2 | 800 |
| West Side No. 2 | Submersible /grinder | 2 | 100 |
| Hazel Street | Submersible | 2 | 150 |

| Table 2-1 cont. | | | |
|---|---|---|---|
| City of Mount Vernon's Sanitary Sewer System Pump Stations | | | |
| **Pump Station** | **Type** | **No. of Pumps** | **Firm Pumping Capacity (gpm)** |
| 19th Street | Submersible | 2 | 280 |
| Division Street | Submersible | 2 | 160 |
| Eaglemont Pump Station No.1 | Submersible | 2 | 560 |
| Eaglemont Pump Station No.2 | Submersible | 2 | 620 |
| South Mount Vernon | Submersible | 2 | |

### Combined Sewer Overflow Structures

Overflows from the combined sewer portions of the City are diverted at three overflow structures to two overflow pump stations. The overflow structures are located at First Street and Freeway Drive, Division Street under the Second Street Overpass, and Park Street at Harrison Street. The overflows from the Freeway Drive and Division Street structures flow together to the Division Street Pump Station. Overflows from the Park Street structure flow to the Park Street Pump Station. The overflow pump stations discharge directly to the Skagit River. A detailed description of the CSO system is presented in Chapter 4.

### Wastewater Treatment Facility

The existing WWTP liquid stream processes consists of coarse bar screens followed by the Influent Pump Station, which pumps to a comminutor. Flows from the West Mount Vernon Pump Station combine with the influent pump station flows at the comminutor and flow through the primary clarifier. The liquid stream continues to the activated sludge pump station, aeration basins, secondary clarifiers, chlorine mixing chamber, chlorine contact basin, and effluent pump station. Effluent is discharged to the Skagit River via a 24-inch outfall.

The existing WWTP solids stream processes consists of primary sludge thickening (via a gravity thickener) and waste-activated sludge thickening (via a dissolved air floatation thickener), anaerobic digestion, biosolids dewatering via belt filter press, and biosolids storage.

# 3. BASIC PLANNING DATA

The basic planning data used to predict the City's future land use and wastewater flows and loads are presented in this chapter. Population growth projections for the City of Mount Vernon from the Office of Financial Management and the urban growth area define the future needs of the City.

## INTRODUCTION

The City of Mount Vernon's current Comprehensive Sewer and Combined Sewer Overflow Reduction Plan was adopted by the City Council in 1994 and approved by the Department of Ecology (DOE) in 1995. In October 1995, a Wastewater Treatment Plant Evaluation was prepared that identified improvements that would be required to provide treatment of combined sewer flows as required by the City's Consent Decree with Department of Ecology. The 1995 report also identified treatment plant improvements required to accommodate growth in the service area. Since the publication of the1995 report, the City has constructed the Kulshan Intercepter and the Central CSO Regulator. This pipeline provides inline storage for combined sewer flows that would have otherwise overflowed to the Skagit River. In November 1998 a Draft Wastewater Flow and Organic Load Projection Report was prepared for the City. At the time the 1998 report was developed, less than a year of operational data from the Central CSO Regulator was available.

The following chapter revises the wastewater flow and load projections for the City based on additional operating data.

## RELATED PLANS

This Comprehensive Sewer Plan Update builds on the previous studies and plans prepared for the City of Mount Vernon, which include:

- 1994 Comprehensive Sewer and Combined Sewer Overflow Reduction Plan
- 1995 City of Mount Vernon Wastewater Treatment Plant Evaluation
- 1998 Wastewater Flow and Organic Load Projection Report
- 2000 Mount Vernon WWTP Mixing Zone Study

## SERVICE AREA CHARACTERISTICS

### Background

Mount Vernon has historically provided sewer service within the Urban Growth Area. Increased conveyance and treatment issues are currently being addressed with this study. Recommended improvements for combined sewer overflow issues are addressed in Chapter 4.

### Geography

The City of Mount Vernon slopes south and west towards the Skagit River. Interstate 5 runs along the western side of the service area. Levees protect the City from flooding by the Skagit River.

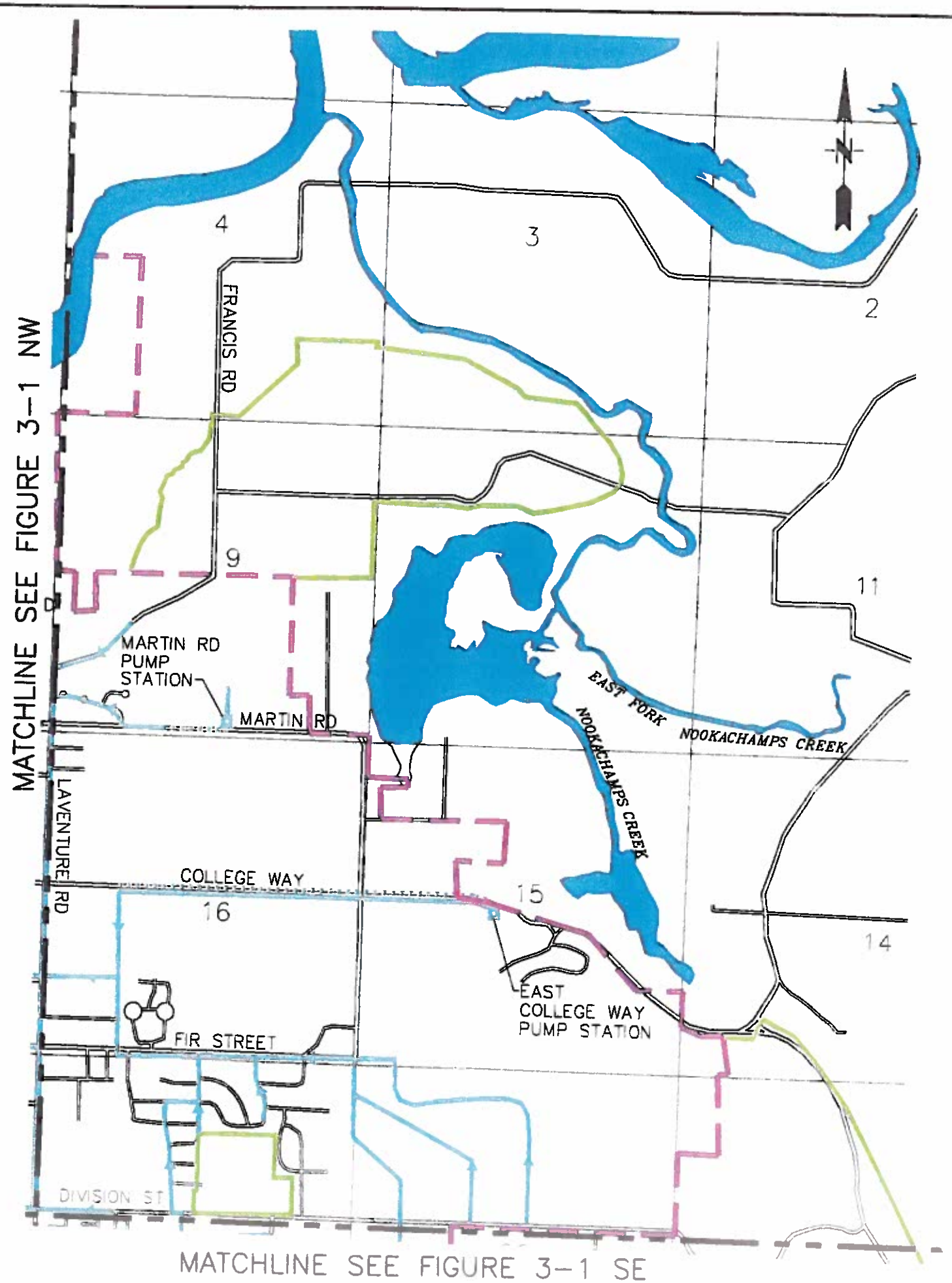
### Existing Sewer Service Area

The existing sewer service area is comprised of connections within the City limits and near future service area. Figure 3-1 delineates the existing sewer service area boundary.

### UGA Sewer Service Area

The planning period for this study is 20 years, with 10- and 20- year projections starting in 2000.

The future sewer service area is the UGA boundary identified by the Skagit County Comprehensive Plan and is delineated graphically in Figure 3-2.




LEGEND:

- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROW)
- EXISTING FORCE MAIN
- EXISTING PUMP STATION
- EXISTING OVERFLOW WEIR
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc

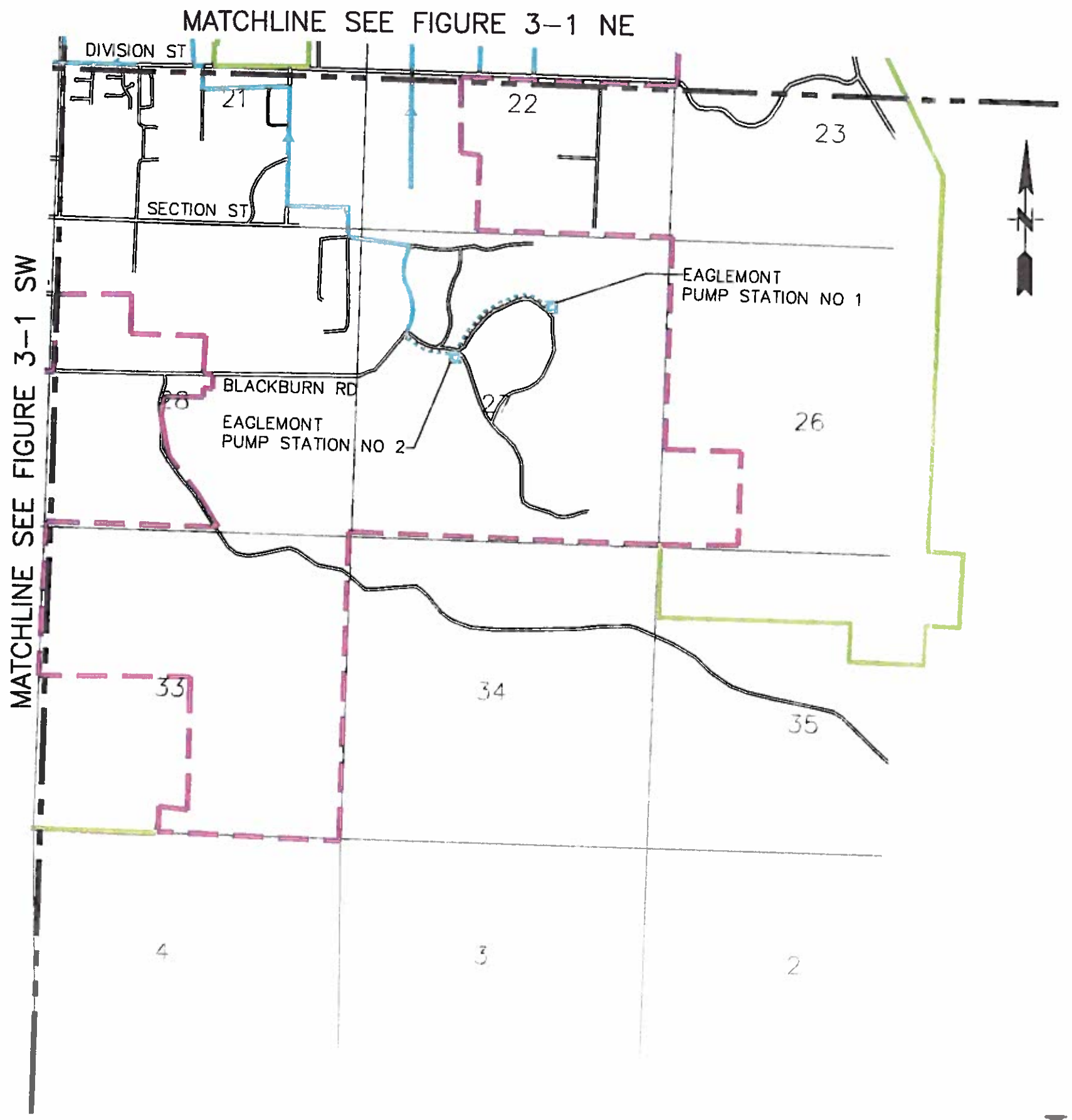
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No: 3-1 NE

DATE 02/17/03




LEGEND:

- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROW)
- EXISTING FORCE MAIN
- EXISTING PUMP STATION
- EXISTING OVERFLOW WEIR
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc.

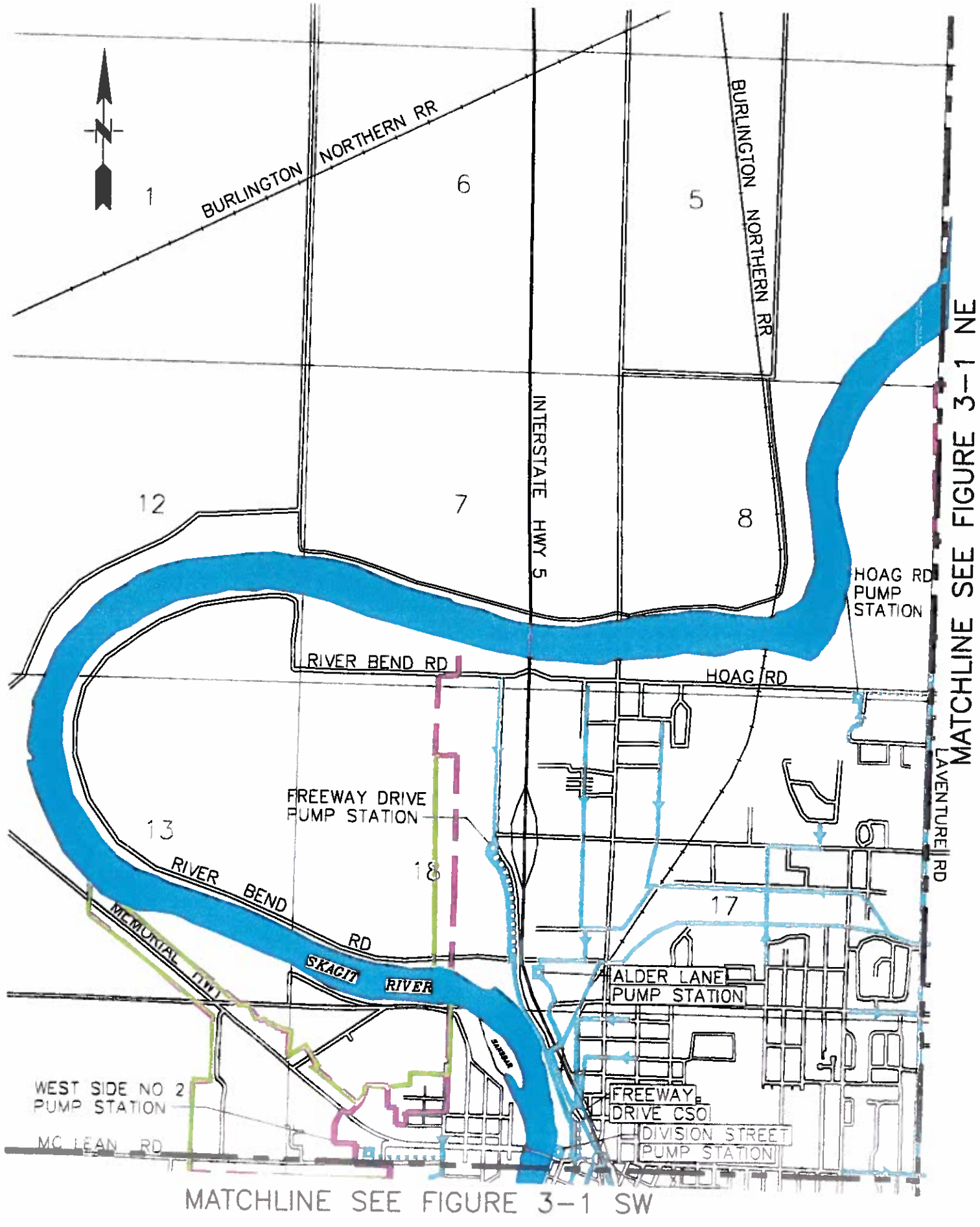
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No: 3-1 SE

DATE: 02/12/03




LEGEND:

- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROW)
- EXISTING FORCE MAIN
- EXISTING PUMP STATION
- EXISTING OVERFLOW WEIR
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc.

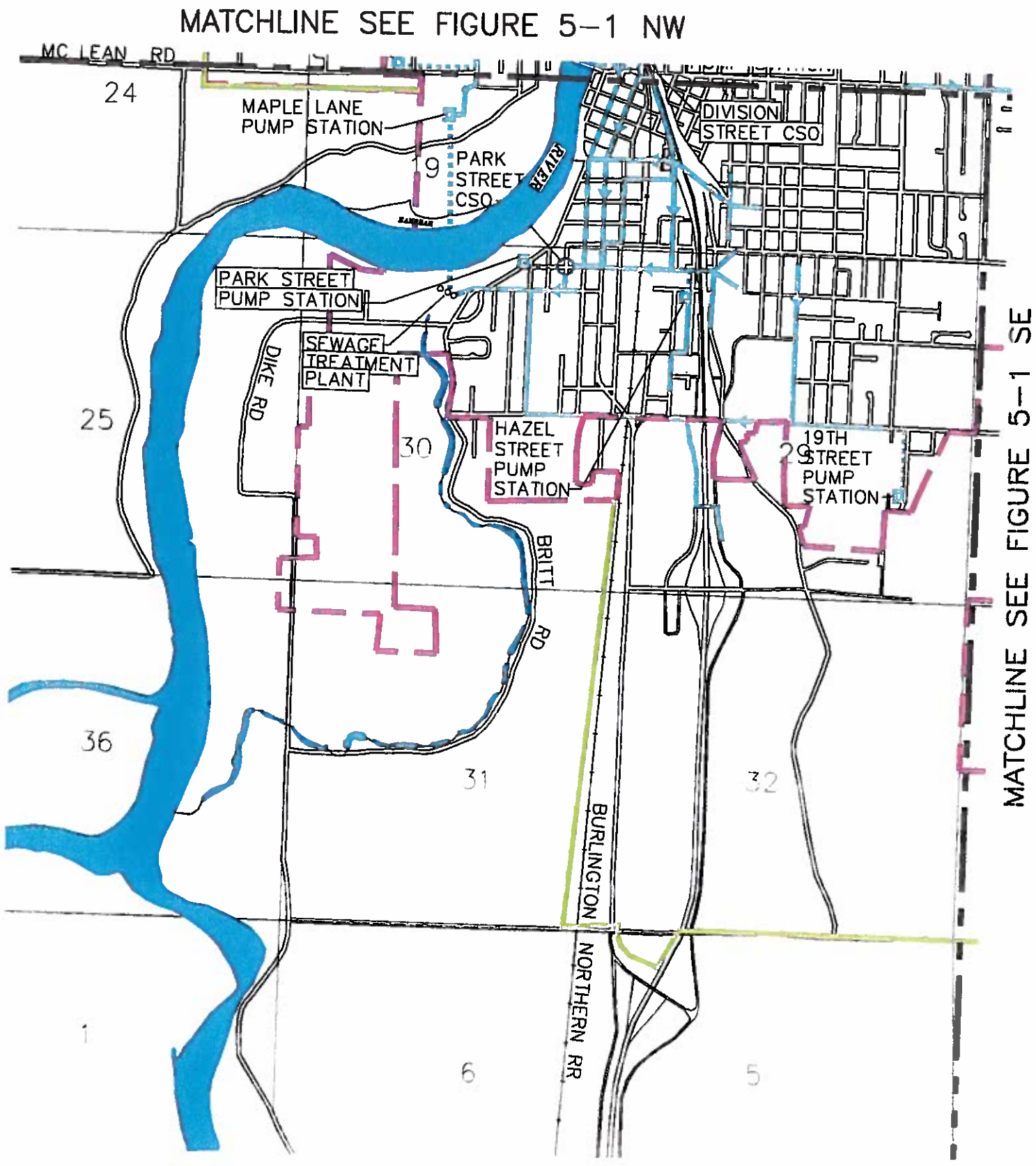
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No: 3-1 NW

DATE: 02/17/03




LEGEND:

- EXISTING MAIN COLL SEWERS (W/ FLOW
- EXISTING FORCE MA
- EXISTING PUMP STA
- EXISTING OVERFLOW
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc

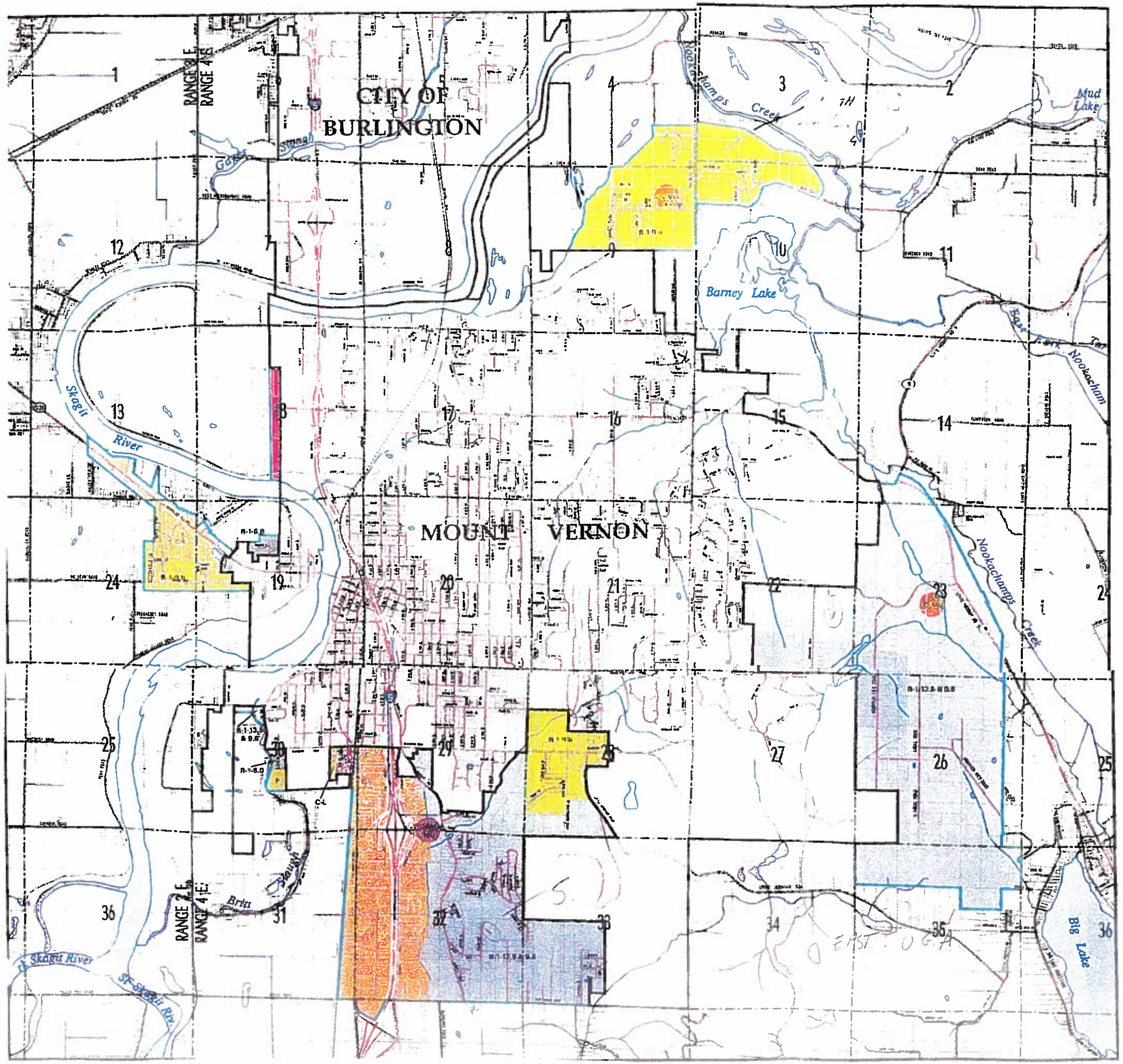
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No: 3-1 SW

DATE: 02/12/03




# LEGEND

- Urban Growth Area
- Commerical / Light Industrial
- Commerical
- Public
- R-1-6.0 *
- Planned Commercial Mixed-Use
- Planned Community Mixed-Use
- Planned Neighborhood Mixed-Use
- R-1-7.6 *
- R-1-9.6 *
- R-1-13.5 & 9.6 *
- Incorporated Areas

* Residential densities are at a minimum 4 dwelling units per acre and a maximum lot size of 1/4 acre (10,890 sq. ft.)

The Skagit County Assessor tax lots depicted on this map represent parcel information as of October 5, 2001. For current up to date parcel information the maps available in the Skagit County Assessor office or on the web at www.skagitcounty.net should be consulted

*August 15, 2001

0 1/2 Mile 1 Mile
Scale in Miles

N
W E
S

*Release date only. This map incorporates offical map changes up to the release date. Changes made between releases are processed and incorporated into the Comprehensive Plan/Zoning maps on the County's website at www.skagitcounty.net and are also hand-recorded on maps at the Skagit County Planning & Permit Center. Please consult the Planning & Permit Center for a record of these changes. For a record of city annexations, contact the Skagit County Auditor's office

SKAGIT COUNTY
GIS
GIS DIVISION

**Figure 3-2**

**CITY OF MOUNT VERNON**

**URBAN GROWTH AREA**

## POPULATION PROJECTIONS

The GMA population projections from the Skagit County Comprehensive Plan for the Mount Vernon Urban Growth Area (UGA) were summarized in the 1998 Wastewater Flow and Organic Load Projection Report. These projections are presented in Table 3-1.

**TABLE 3-1**

| City of Mount Vernon Population Projections and Service Area Population Projections | | |
|---|---|---|
| **Year** | **City of Mount Vernon GMA Population Projections** | **City of Mount Vernon Service Area Population Projections** |
| 1995 | 23,416 | |
| 1998 | 26,485 (interpolated) | 22,540 |
| 2000 | 28,531 | 26,232 |
| 2005 | 33,463 | 29,431[1] |
| 2010 | 38,396 | 35,861[1] |
| 2015 | 43,559 | 42,292[1] |
| 2020 | 48,722[1] | 48,722[2] |

1. Extrapolated from GMA Projections
2. All areas within the GMA are served by 2020

The study noted that the 1998 interpolated population was greater than the population of 22,540 used by the Washington State Department of Revenue. The discrepancy was attributed to the fact that areas within the UGA that are not currently incorporated in the City limits. For wastewater planning purposes it is assumed that all future areas within the UGA will be annexed and the City will provide wastewater service to the projected GMA population by the year 2020. For purposes of estimating current loads, the 2000 population is assumed to be 23,000.

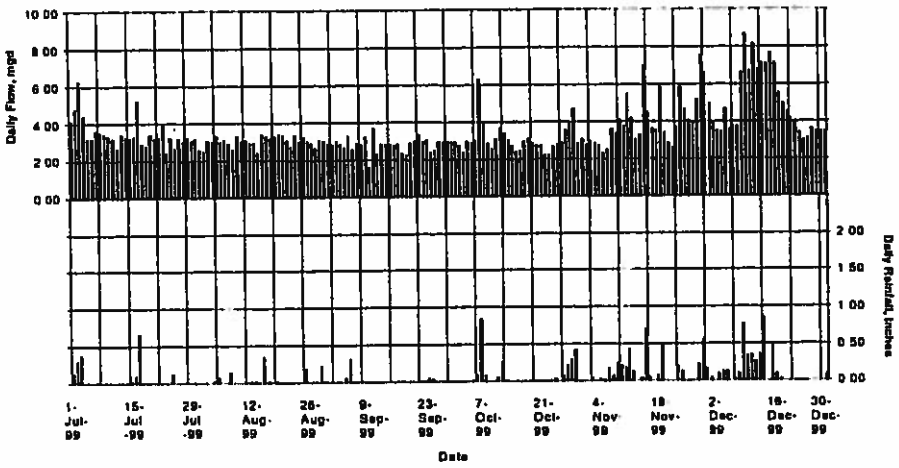
## HISTORICAL FLOWS AND LOADS

### Wastewater Treatment Plant Flow

Wastewater treatment plant daily flow records from the last five years were reviewed to determine the historical loading. The flow records were compared with daily rainfall to determine the impact of rainfall on plant flows. The rainfall is measured at the wastewater treatment plant. Figure 3-3 illustrates the daily flows with the recorded rainfall for July 1 to

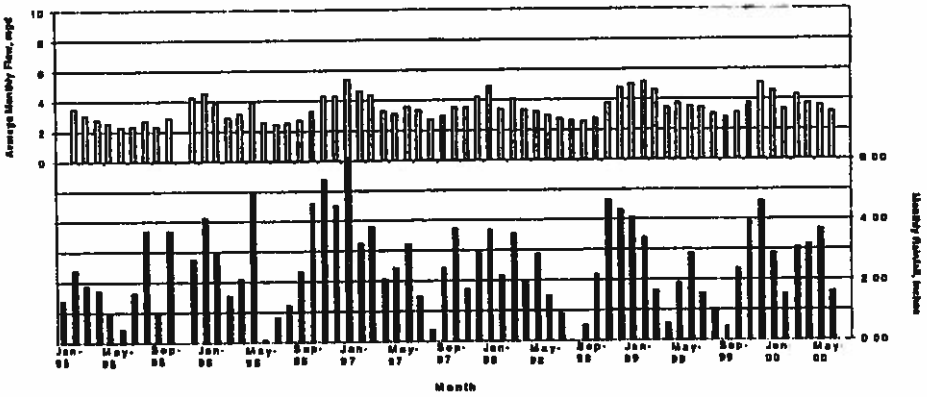
December 31, 1999. This plot illustrates that during late summer the flows reach a base rate of about 2.6 mgd. The plot also illustrates in the dry weather period the five day work week of Draper Valley Farms(DVF), Inc, which discharges from 0.4 to 0.6 mgd when in operation. In November and December rain caused the direct increase in treatment plant flows.

**Figure 3-3 Mount Vernon Daily WWTP Flows and Rainfall, July 1 - December 31, 1999**




The seasonal trend in flow is observed when average monthly flows are plotted against rainfall as shown in Figure 3-4.

**Figure 3-4 City of Mount Vernon Monthly WWTP Flows**




### Commercial Flow

The 1998 Wastewater Flow and Organic Load Projection Report estimated 0.6 mgd of flow from 638 commercial customers based on water meter readings. Skagit County

documented that the existing commercial area in Mount Vernon is 292 acres. The existing commercial loading rate is 2,055 gpd per acre.

**Industrial Flow**

The major industrial wastewater discharger in Mount Vernon is Draper Valley Farms, Inc. (DVF), a chicken processing facility. The current wastewater discharge, on a monthly basis, is approximately 0.45 mgd.

**Domestic Flow**

The remaining dry weather flow component after commercial and industrial flows are removed is domestic sanitary flow. The existing domestic flow is estimated as follows:

| | | |
|---|---|---|
| Total Dry Season Flow | | 2.62 mgd |
| | Commercial Flow | - 0.60 mgd |
| | Industrial Flow | - 0.43 mgd |
| Total | Domestic Flow | 1.59 mgd |

Based on an estimated population of 23,000, the current per capita loading rate without infiltration and inflow is 69 gpcd (1.59 mgd/23,000).

**Infiltration & Inflow**

As rainfall increases there is a corresponding increase in wastewater flows. This extraneous flow is known as infiltration and inflow. Inflow is a direct entry of storm water into the sewer system through direct piping connections such as catch basins, leaking manhole covers, roof gutters, driveway drains and other area drains.

Infiltration is ground water that enters the sewer system through defects or other subsurface connections. Infiltration sources include cracks in pipes, manholes, subsurface foundation drains or even basement and crawl space sump pumps. During heavy rains infiltration may increase rapidly and in a review of flow data this rain induced infiltration may appear to be inflow.

The older portions of Mount Vernon have combined sewers. These sewers were originally designed to convey both storm and sanitary sewer flows. Many parts of the separated system also experience infiltration and inflow.

In addition to the storm water inflow component, these portions of the system are constructed of clay and concrete pipe. Due to their age, materials, and methods of construction, these portions of the system are subject to higher levels of infiltration and inflow. To determine the 'additional infiltration and inflow component,' an evaluation was made to quantify this component. This was computed by subtracting the commercial, industrial, and residential flow components from the maximum monthly flow. The DOE guidelines of 100 gpcd for new sewer systems (including infiltration and inflow) was used to establish the baseline residential flow rate. The 'additional infiltration and inflow component' was then computed as follows:

| | |
|---|---|
| Maximum Month Flow (January 1997) | 5.39 mgd |
| Commercial Flow | - 0.60 mgd |
| Industrial Flow (DVF) | - 0.43 mgd |
| Baseline Residential Flow [23,000 persons x 100 gpcd[1]] | - 2.30 mgd |
| Additional Infiltration and Inflow Component | 2.06 mgd |

1. DOE criteria includes normal Infiltration and inflow for a separated sanitary system.

There could be a deterioration of the system that could result in additional infiltration and inflow into the system. However, it is also anticipated that reconstruction of sewers will separate inflow sources and reduce infiltration. For the purposes of planning it is assumed that the current infiltration and inflow rate will remain the same throughout the planning period and improvements will offset infiltration and inflow for the existing system.

### Combined Sewer Flows

Mount Vernon has combined sewers in the older portions of the City. The storm drainage connections produce excess flow during storm events. Combined Sewer Overflow (CSO) structures allow flow in excess of the sewer system and treatment plant capacity to be discharged directly to the Skagit River.

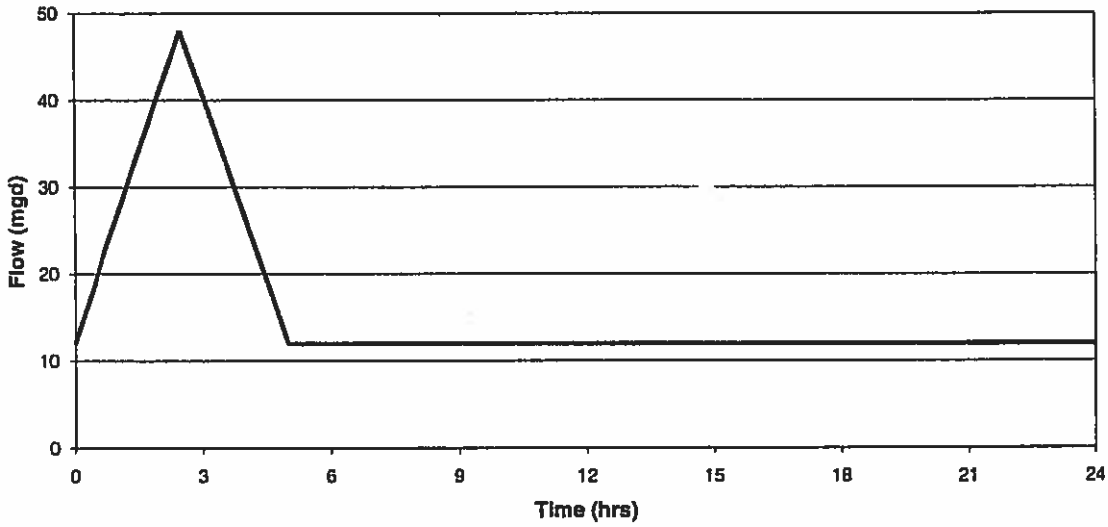
The CSO Baseline for Mount Vernon was established in 1988. It predicted an annual CSO volume of 116.5 MG for the average annual rainfall of 31.95 inches. Based on the 1988 collection system there was a 95 percent confidence that the volume, with an average annual rainfall, would be between 92 MG and 141 MG.

During 1988, flow monitoring allowed determination of not only the CSO Baseline, but also the peak flow rate due to combined flows. During some of the periods of high flow, the peak flow rates were not recorded, but estimates made in the 1994 Comprehensive Sewer and Combined Sewer Overflow Reduction Plan predicted the peak system flow rate at 45 to 50 mgd. In 1997, the City placed the Central CSO Regulator, a 60-inch diameter interceptor, on-line. This has significantly reduced the occurrences of combined sewer overflows. A detailed summary and analysis of recent combined sewer history is presented in Chapter 4.

The May 16, 1988, storm event was estimated to be approximately a two-year storm recurrence. It was selected as a design storm event, and was considered to be reasonably conservative. In the 1995 Wastewater Treatment Plant Evaluation, the peak flow for the May storm event was estimated to be 47 mgd. Combining this flow with the one mgd contributed by the West Mount Vernon Pump Station yields a peak system flow rate of 48 mgd. The affects of the Central CSO Regulator are analyzed in Chapter 9.

Compliance with the DOE consent decree will require limiting untreated overflows to one event per year. To estimate the volume of the stormwater component for the one year storm event, historical CSO data was reviewed. The largest recorded overflow was on May 16, 1988. In the 1995 Wastewater Treatment Plant Evaluation, a detailed analysis of this storm was performed. An idealized combined sewer flow hydrograph was created in that evaluation and is presented in Figure 3-5.

**Figure 3-5 Idealized Combined SewerFlow Hydrograph - May 16, 1988**




The idealized combined sewer flow hydrograph shows a combined peak flow rate of 48 mgd. The maximum day storm flow component (total volume of storm flow) can be estimated from this hydrograph. The historic maximum day sanitary flows are subtracted from the total volume of flow in 24 hours to obtain the storm flow component as follows:

| | |
|---|---|
| Total Combined Sewer Flows | 15.8 mg |
| Historical Sanitary Maximum Day Flow | 9.2 mg |
| Storm Flow Component | 6.6 mg |

The BOD and TSS loads for the storm flow component were estimated by reviewing existing data for CSO events. BODs for the larger storm events typically ranged from 10 to 60 mg/L, and TSS typically ranged from 20 to 100 mg/L. The maximums were applied to the estimated flows to establish the maximum anticipated loads. These are summarized in Table 3-2.

**Table 3-2**

| Combined Sewer Component Flow and Load Projections for 2020 | | |
|---|---|---|
| **Component** | **Storm Maximum Day[1]** | **Peak Hour[2]** |
| Flow (mgd) | 6.6 mgd | 48 mgd |
| BOD (ppd) | 3,300 ppd | - |
| TSS (ppd) | 5,500 ppd | - |

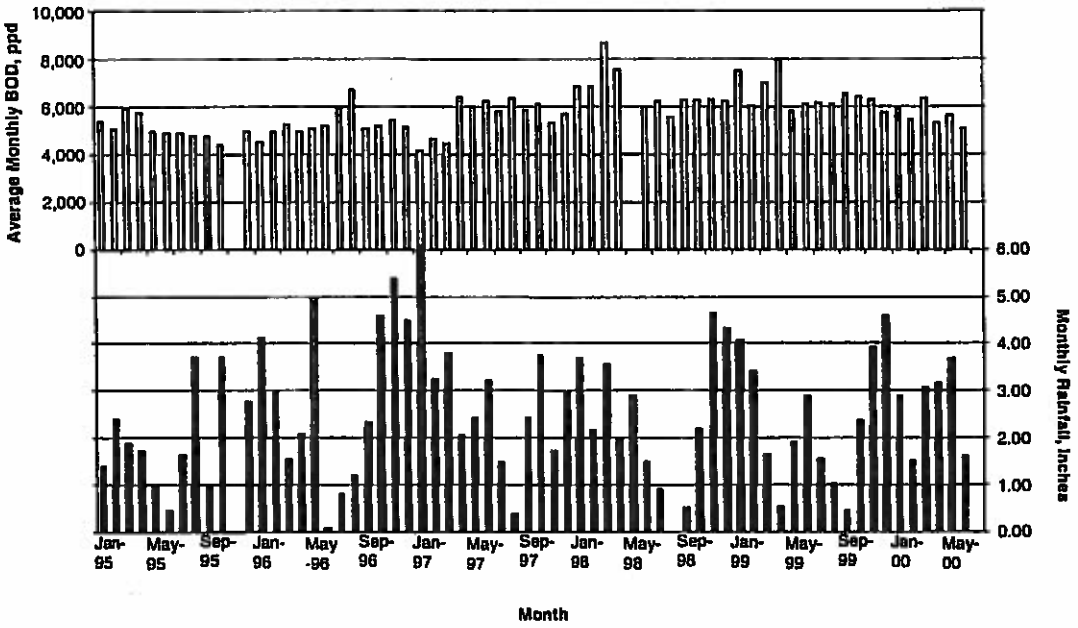
1. Storm flow component estimated from May 16, 1988, storm event.
2. Sanitary and storm components combined flow estimates

## Treatment Plant Loading

### Biochemical Oxygen Demand (BOD)

Figure 3-6 illustrates the monthly average day BOD loading to the treatment plant from January 1995 to June 2000. There are spikes in the BOD in March and April 1998 and in January, March, and April of 1999. A review of the daily treatment plant data determined that the averages of these months were significantly impacted by one or two days where the reported BOD load to the plant was 10,000 to 20,000 pounds per day. The treatment plant staff noted that there is a sampling problem that occurs during periods of high rainfall that caused the measured BOD concentration of the influent to be higher than actual loads. This assumption was verified by reviewing the BOD concentrations from the effluent from the primary clarifier for these days. Based on this analysis the monthly BOD load to the treatment plant is approximately 6,400 pounds per day.

## Figure 3-6 City of Mount Vernon Monthly BOD Loadings




The BOD loading at the plant does not show any correlation with rainfall and the BOD load appears to remain relatively constant year round.

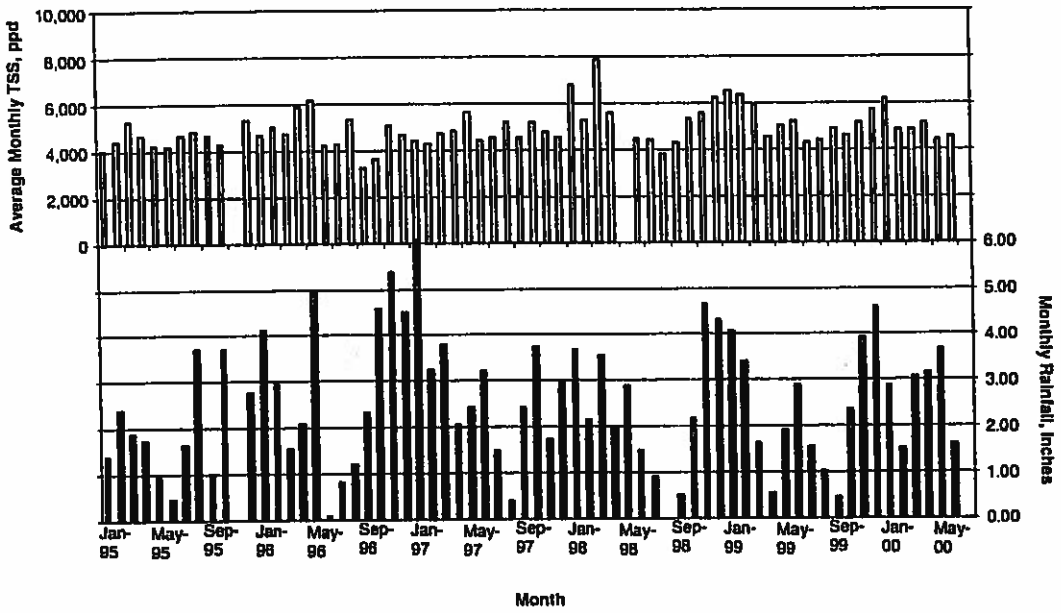
The only current industrial customer, Draper Valley Farms, Inc. (DVF), discharges between 500 and 1,200 ppd of BOD per month. Additional information on these loads Is provided in Chapter 6. Based on previous City discussions with DVF it is assumed that the future flows from the plant could approach 0.75 mgd (Appendix A). Future BOD loads were estimated by increasing from the current discharge permit levels of 1,300 lbs. per day to 1,550 lbs. per day to allow for the increased flows.

The Maximum Month Average Day BOD load to the treatment plant from domestic and commercial sources is approximately 7,900 ppd, without Draper Valley Farms, Inc. This may be due to non-representative samples within the Central Interceptor after a storm event.

### Total Suspended Solids (TSS)

Figure 3-7 provides the monthly average day TSS compared with rainfall. The reported TSS loads to the plant in March and December 1998 and in January 1999 through March 1999 were affected by a few days with excessive loads.

## Figure 3-7 City of Mount Vernon Monthly TSS Loading




The review of TSS load and rainfall does not appear to show a correlation; however, there likely is some additional solids loading to the plant associated with the first flush of the system with rainfall in the Fall or following an extended dry period. Otherwise, the TSS load appears to remain relatively constant year round. The monthly TSS load to the treatment plant is approximately 5260 ppd.

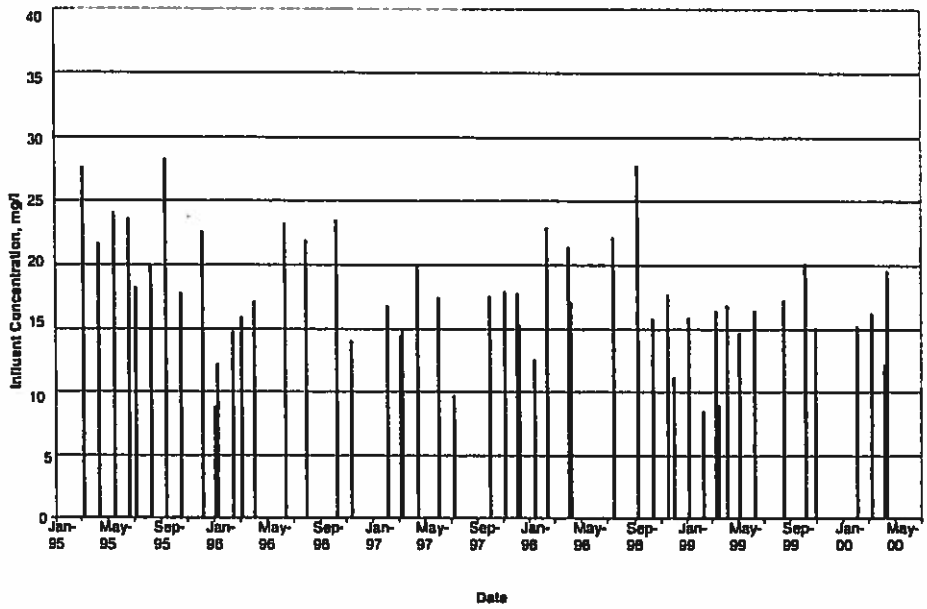
The TSS load from DVF is typically from 400 to 600 ppd based on an influent concentration of 125 to 150 mg/L. The industrial component for DVF is further reviewed in Chapter 6.

The Maximum Month Average Day TSS load to the treatment plant from domestic and commercial sources is approximately 7,600 ppd, without DVF. This may also be due to settlement of solids and non-representative samples within the Central Interceptor after a storm event.

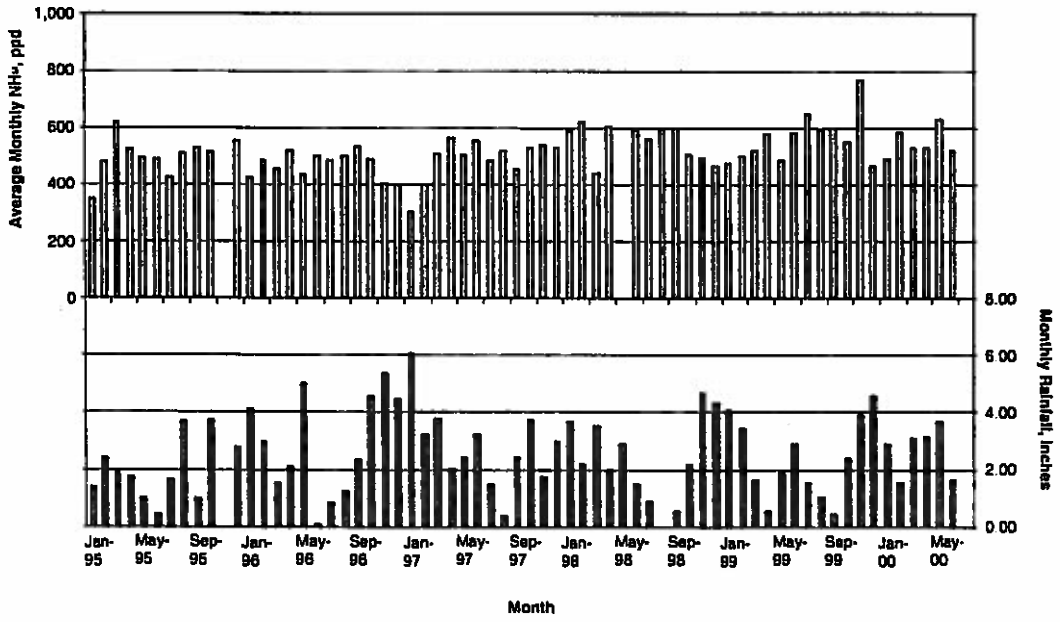
### Ammonia

The historical influent ammonia concentration typically ranged between 10 to 30 mg/L as seen in Figure 3-8. The ammonia loading to the plant in pounds per day is illustrated in Figure 3-9. Similar to BOD and TSS loadings, the total ammonia load to the plant does not seem to be related to rainfall and appears to remain constant through the year. The average month ammonia load to the plant is approximately 550 pounds per day.

## Figure 3-8 City of Mount Vernon Ammonia Nitrogen Influent Concentration




## Figure 3-9 City of Mount Vernon Monthly Ammonia Loading




The 1998 Wastewater Flow and Organic Load Projection Report estimated the average daily ammonia concentration from DVF at 22 mg/L. This equates to a total daily load of approximately 84 ppd. The domestic and commercial ammonia load would be 466 ppd.

### Summary of Historical Flows and Loads

Table 3-3 summarizes the historical flows for the City. Table 3-4 summarizes the historical loads for the City.

**Table 3-3**

| Historical Flows for the City of Mount Vernon | |
|---|---|
| **Parameter** | **Historical Flow** |
| Per Capita Flow[1] | 69 gpd |
| Commercial Flow | 2,055 gpad |
| Draper Valley Flow | 0.46 mgd |
| Average Annual Day (AAD) | 3.7 mgd |
| Average Day Maximum Month (ADMM) | 5.4 mgd |
| Maximum Day | 9.2 mgd |

1. Does not include infiltration and inflow

**Table 3-4**

| Historical Average Month Loads for the City of Mount Vernon | | | |
|---|---|---|---|
| **Parameter** | **Historical BOD** | **Historical TSS** | **Historical $NH_3$-N** |
| Domestic and Commercial Loading | 5,200 ppd | 4,600 ppd | 370 ppd |
| Domestic and Commercial Per Capita Loading | 0.18 ppd/capita | 0.16 ppd/capita | 0.016 ppd/capita |
| Commercial Loading w/o Domestic | 1,000 ppd[1] | 1,000 ppd[2] | 100 ppd[3] |
| Industrial Loading (Draper Valley) | 1,200 ppd[4] | 660 ppd[5] | 84 ppd |
| Industrial Concentration (Draper Valley) | 300 mg/L[4] | 160 mg/L[5] | 22 mg/L |
| Total WWTP Loading | 6,400 ppd | 5,260 ppd | 554 ppd |

1. Based on 0.6 mgd and BOD concentrations of 200 mg/L
2. Based on 0.6 mgd and TSS concentrations of 200 mg/L
3. Based on 0.6 mgd and NH3 concentrations of 20 mg/L
4. October 1999
5. July 1999

## PROJECTED FLOWS AND LOADS

Projected flows and loads were developed based upon DOE criteria and historical patterns for the City.

BOD load projections were developed independently for both domestic and commercial flow components. The historical domestic BOD loading has been 0.18 ppcd. This was increased to 0.20 ppcd for future predictions and matches DOE design criteria to be used when this information is not available. Domestic loads were based on 0.20 lbs. per capita per day. Commercial loads were based on a BOD concentration of 200 mg/L.

Similar to the BOD loadings, the TSS load projections were based upon 0.20 ppd/capita for residential loads and commercial contributions of 200 mg/L.

$NH_4$-N load projections were based upon 0.016 ppd/capita for residential and 20 mg/L for commercial and industrial contributions. Draper Valley Farms, Inc.'s contribution was based on a concentration of 22 mg/L.

### Wastewater Treatment Plant Flow

Future flow projections for 2010 and 2020 are based on the estimated population, projected DVF flows, and the future commercial and other industrial loads. This information was obtained from the Skagit County Comprehensive Plan and the 1998 Wastewater Flow and Organic Load Projection Report. Flows from other industrial areas are based on the same flow rate as commercial flow. The future flow projections for these sources are summarized in Table 3-5.

The treatment plant has experienced a maximum influent flow rate of 14.8 mgd which is about 20 percent in excess of the existing peak hour design flow rate. Since the State WAC for CSO reduction requires CSO agencies to maximize the flow to the secondary plant and since the Central CSO regulator provides equalizing storage upstream of the plant it is possible that the treatment plant will experience the peak hydraulic capacity for periods exceeding one day.

Table 3-5

| Flow Projections for the City of Mount Vernon | | | | | |
|---|---|---|---|---|---|
| | 2010 Projection | 2020 Projection | Flow Rate | 2010 Flows | 2020 Flows |
| Residential Population | 35,861 | 48,722 | 100 gpcd | 3.59 mgd | 4.87 mgd |
| Commercial Area | 500 ac | 660 ac | 2,055 gpad | 1.03 mgd | 1.36 mgd |
| Draper Valley Farms, Inc. | 0.75 mgd | 0.75 mgd | - | 0.75 mgd | 0.75 mgd |
| Other Industrial Area | 337 ac | 446 ac | 2,055 gpad | 0.69 mgd | 0.92 mgd |
| Base System Flow | | | | 6.06 mgd | 7.90 mgd |
| Additional Inflow and Infiltration Component (ADMM) | | | | 2.03 mgd | 2.03 mgd |
| ADMM Flow | | | | 8.09 mgd | 9.93 mgd |
| Peak Hour Flow[1] | | | | 14.9[2] mgd | 18.3[3] mgd |

1. Peaking factor based on L.A. Peaking Curve, Appendix B.
2. Peaking factor of 2.13.
3. Peaking factor of 2.06.

### Organic Loads

Future load projections for 2010 and 2020 are based on the estimated population and future commercial and industrial loads and the projected Draper Valley Farms, Inc. loads. The future projections for these sources are summarized in Table 3-6 to Table 3-8.

Table 3-6

| Projected BOD Loadings for the City of Mount Vernon | | | | | |
|---|---|---|---|---|---|
| Load Source | Projected Population/Flow | | Average Daily Loading | Projected Loads | |
| | 2010 | 2020 | | 2010 | 2020 |
| Residential Population | 35,861 | 48,722 | 0.20 ppd/capita | 7,170 ppd | 9,740 ppd |
| Commercial | 1.03 mgd | 1.36 mgd | 200 mg/L | 1,720 ppd | 2,270 ppd |
| DVF | 0.75 mgd | 0.75 mgd | 250 mg/L | 1,550 ppd[1] | 1,550 ppd[1] |
| Other Industrial | 0.69 mgd | 0.92 mgd | 200 mg/L | 1,150 ppd | 1,540 ppd |
| Total | | | | 11,590 ppd | 15,100 ppd |

1. Based on existing discharge permit limit of 1,300 ppd increased by 19% anticipated hydraulic increase provided by DVF.

Table 3-7

| Projected TSS Loadings for the City of Mount Vernon | | | | | |
|---|---|---|---|---|---|
| Load Source | Projected Population/Flow | | Average Daily Loading | Projected Loads | |
| | 2010 | 2020 | | 2010 | 2020 |
| Residential Population | 35,861 | 48,722 | 0.20 ppd/capita | 7,172 ppd | 9,744 ppd |
| Commercial | 1.03 mgd | 1.36 mgd | 200 mg/L | 1,720 ppd | 2,270 ppd |
| DVF | 0.75 mgd | 0.75 mgd | | 890 ppd[1] | 890 ppd[1] |
| Other Industrial | 0.69 mgd | 0.92 mgd | 200 mg/L | 1,150 ppd | 1,540 ppd |
| Total | | | | 10,932 ppd | 14,444 ppd |

1. Based on existing discharge permit limit of 750 ppd increased by 19% anticipated hydraulic increase provided by DVF.

Table 3-8

| Projected $NH_4$-N Loadings for the City of Mount Vernon[1] | | | | | |
|---|---|---|---|---|---|
| Load Source | Projected Population/Flow | | Average Daily Loading | Projected Loads | |
| | 2010 | 2020 | | 2010 | 2020 |
| Residential | 35,861 | 48,722 | 0.016 ppd/capita | 574 ppd | 780 ppd |
| Commercial | 1.03 mgd | 1.36 mgd | 20 mg/L | 172 ppd | 227 ppd |
| Other Industrial | 0.69 mgd | 0.92 mgd | 20 mg/L | 115 ppd | 154 ppd |
| DVF | 0.75 mgd | 0.75 mgd | 22 mg/L | 138 ppd | 138 ppd |
| Total | | | | 999 ppd | 1,299 ppd |

1. $NH_4$-N loading based on influent only. Additional $NH_4$-N loading to secondary treatment process by internal recycle of anaerobic digester supernatant.

## SUMMARY OF PROJECTED FLOWS AND LOADS

The flow and loading projections for the treatment plant were developed in the previous section. These flows and loadings are summarized in Table 3-9. For maximum day and peak hour loadings, concentrations were assumed and loadings were calculated as shown.

Table 3-9

| WWTP and CSO Flow and Load Projections | | | | |
|---|---|---|---|---|
| Year | Parameter | Average Day Maximum Month | Maximum Day | Peak Hour |
| 2010 | Flow (mgd) | 8.1 | 11.4 | 14.9 |
| 2010 | BOD (ppd) | 11,590 | 14,311 | - |
| 2010 | TSS (ppd) | 10,932 | 13,500 | - |
| 2010 | NH4-N (ppd) [1] | 999 | 1,040 | - |
| 2020 | Flow (mgd) | 9.9 | 13.9 | 18.3 |
| 2020 | BOD (ppd) | 15,100 | 17,338 | - |
| 2020 | TSS (ppd) | 14,444 | 16,600 | - |
| 2020 | NH4-N (ppd)[1] | 1,299 | 1,261 | - |
| 2020 | CSO Flow (mgd) | - | 6.6[2] | 48[3] |
| 2020 | CSO BOD (ppd) | - | 3,300 | - |
| 2020 | CSO TSS (ppd) | - | 5,500 | - |

1. $NH_4$-N loading based on influent only. Additional $NH_4$-N loading to secondary treatment process by internal recycle of anaerobic digester supernatant.
2. Storm flow component estimated from May 16, 1988, storm event.
3. Total of sanitary and storm component flow estimates

# 4. COMBINED SEWER SYSTEM

## INTRODUCTION

The State of Washington requires agencies with combined sewers to reduce untreated combined sewer overflows to an average of one event per year. The City of Mount Vernon developed a two phase CSO reduction plan and subsequently entered into a consent decree with the Department of Ecology. The first phase required the City to construct the Central CSO Regulator by December 2000. The second phase requires the City to construct treatment facilities by January 2015 that will reduce the remaining CSOs to one untreated event per year. The Central CSO Regulator was constructed and placed into service December 1997.

## TOTAL MAXIMUM DAILY LOAD (TMDL)

The Lower Skagit River has a TMDL limit for both dissolved oxygen (DO) and fecal coliform (see Chapter 7). The limits for DO will not apply during CSO events. The TMDL for fecal coliform will apply to CSOs, but will be determined as a geometric mean. This allows the City of Mount Vernon to have one untreated CSO event per year and remain in compliance. In effect, the TMDL, with regard to CSO events, will be met when all treated CSO flows meet the technology based limits of the NPDES permit (400 cfu/100 mL weekly average) and untreated CSOs are reduced to an average of one event per year.

## EXISTING CSO SYSTEM

### Combined Sewer System

The existing sewer system consists of both sanitary and combined sewers. The combined sewer lines were primarily constructed prior to 1960. They serve approximately 555 acres in the older and downtown areas of Mount Vernon. Flows from the combined area are conveyed to the WWTP, with overflows being conveyed to two pump stations through three overflow structures:

- Freeway Drive Overflow Structure conveys flow to the Division Street Pump Station;
- Division Street Overflow Structure conveys flow to the Division Street Pump Station; and
- Park Street Overflow Structure conveys flows to the Park Street Pump Station.

### Combined Sewer Overflow Pump Stations

Two pump stations convey combined sewer overflows to the Skagit River. Table 4-1 describes these pump stations.

**Table 4-1**

| City of Mount Vernon's Combined Sewer Overflow Pump Stations | | | |
|---|---|---|---|
| **Pump Station** | **Type** | **No. of Pumps** | **Pumping Capacity[1]** |
| Division Street | Mixed Flow Vertical | 3 | 22,300 |
| Park Street | Wetwell/Drywell Horizontally Mounted Centrifugal | 4 | 5,400 gpm[2] |

1. Design pumping rate for all pumps operating.
2. An emergency backup unit is available, with a maximum capacity of approximately 6,500 gpm.

### Central CSO Regulator

The Central CSO Regulator is a 60-inch diameter pipeline in downtown Mount Vernon. It provides conveyance and storage of combined and sanitary flows. During dry weather, wastewater flows are conveyed to the WWTP with the CSO Regulator acting as a gravity sewer pipe. During wet weather, the CSO Regulator is designed to store CSOs in the pipe, rather than discharging them to the Skagit River, and convey the wastewater to the WWTP as capacity becomes available. The CSO regulator provides approximately 1.1 million gallons of in-line storage and consists of:

- 6,800 feet of 60-inch concrete pipe;
- 600 feet of 30-inch concrete pipe;
- One flow regulating structure;
- Three flow control structures;
- Three overflow structures; and
- One Valve Structure on Cameron Way.

The CSO regulator is divided into five storage reservoirs, with storage volumes of 200,000 gallons, 197,000 gallons, 287,000 gallons, 285,000 gallons, and 131,000 gallons, for a total storage capacity of 1.1 million gallons.

### Central CSO Regulator Hydraulic Performance

The Central CSO Regulator provides conveyance capacity to the wastewater treatment plant for combined sewer flows. The pipeline includes structures that allow excess volume of the pipeline to be used for inline storage of combined sewage. The 1995 Comprehensive Sewer and Combined Sewer Reduction Plan anticipated a reduction of overflows to an estimated 12 events per year.

Since the Central CSO Regulator was placed into service in December 1997, the number of overflow events has been reduced to approximately 8 events per year. The overflows that were documented from November 1998 to August 2000 are summarized in Table 4-2.

**Table 4-2**

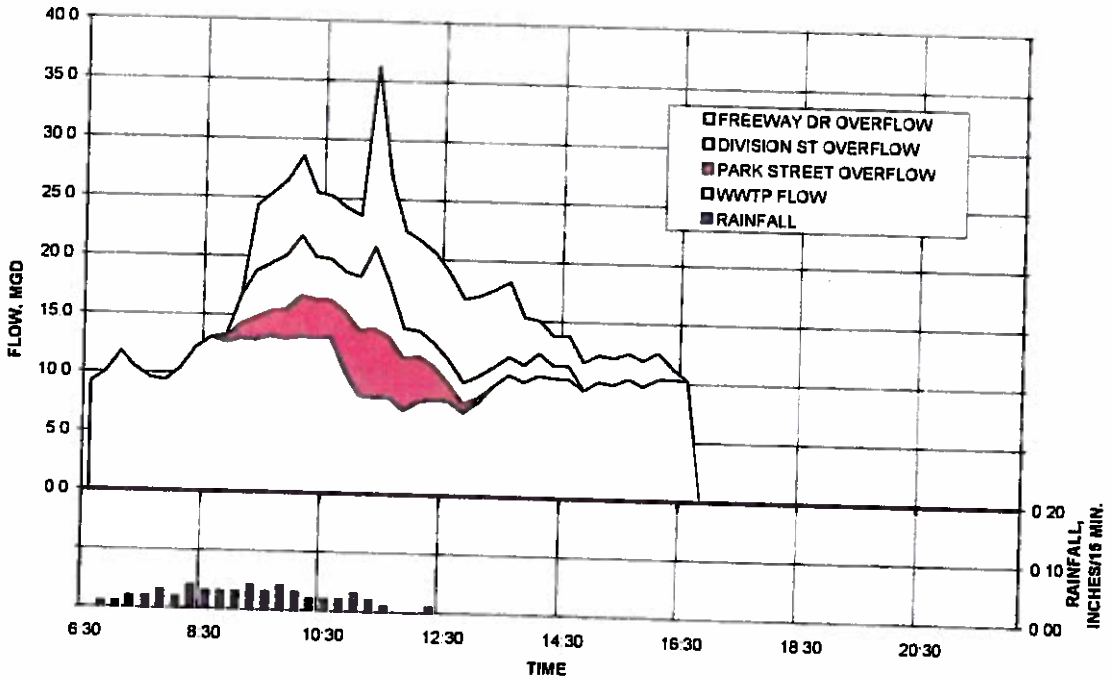
**Combined Sewer Overflows from November 1998 to 2000**

| Date of Overflow | Overflow Volume, gal. | Peak System Flow Rate, mgd[2] | Range of TSS Concentration, mg/L | Range of BOD Concentration, mg/L |
|---|---|---|---|---|
| Nov 13, 1998 | 364,000 | 18.5 | 39 - 68 | 18 - 57 |
| Dec 29, 1998 | 1,845,000 | 36.2 | 45 - 84 | 19 - 27 |
| Jan 10, 1999 | 2,303,000 | 27.7 | 14 - 39 | 6 - 33 |
| Jan 14, 1999 | 388,000 | 14.0 | 22 - 96 | 9 - 53 |
| May 7, 1999 | 44,000 | 16.5 | 44 - 54 | 6 |
| Jun 24, 1999 | 999,000 | 31.0 | 48 - 285 | 9 - 41 |
| Jan 25, 2000 | 906,000 | 21.8 | 46 - 77 | 21 - 50 |
| Apr 13, 2000[1] | 9,624,000 | 32.3 | N/A | N/A |
| Aug 18, 2000 | 396,000 | 17.4 | 111 - 119 | 3 - 4 |

1. The April 13, 2000 event has estimated flow data and TSS and BOD data were not available due to an equipment failure.
2. The Peak System Flow Rate includes all system flows including the wastewater treatment plant flow and overflows at Park Street Pump Station and Division Street Pump Station.

A cumulative flow hydrograph of the December 29, 1998 overflow event is illustrated in Figure 4-1. This figure illustrates the total sewer system flows including the wastewater treatment plant, overflows at the Park Street Pump Station, and overflows at the Division Street Pump Station.

**Figure 4-1 City of Mount Vernon Combined Sewer System Flows, Cumulative Flows for December 29, 1998**



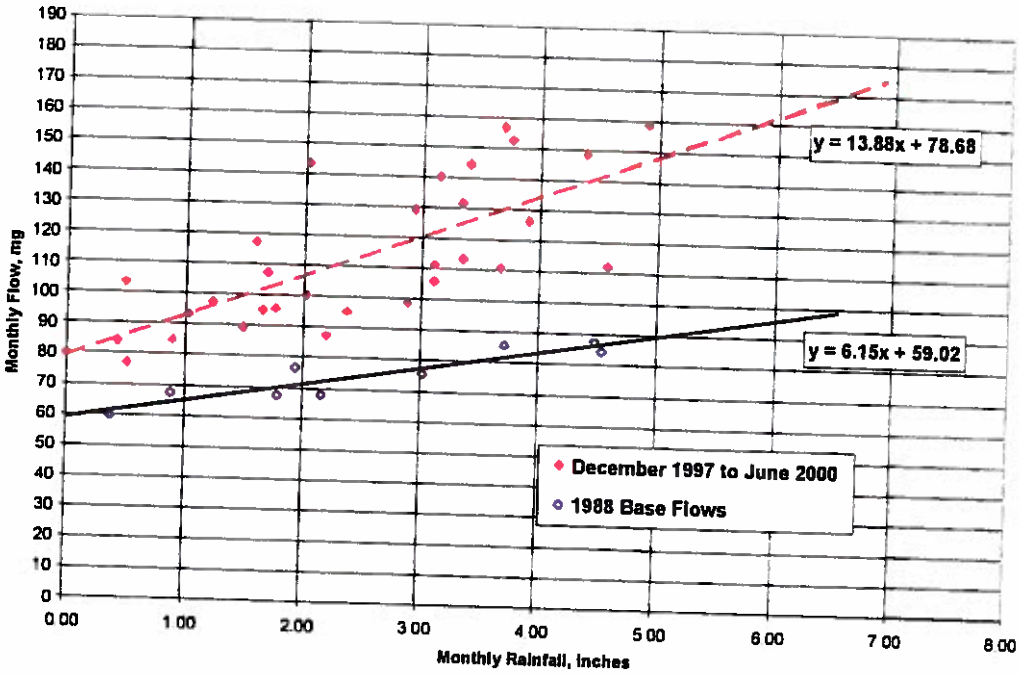

The October 1995 Wastewater Treatment Plant Evaluation evaluated the facilities required based on a peak design system flow rate of 48 mgd. The peak system flow rate observed since the Central CSO Regulator has been in service was 36.2 mgd. A detailed evaluation of the return frequency of this flow rate has not been performed.

For planning purposes it is recommended that 48 mgd continue to be used for a peak system flow rates.

### Central CSO Regulator Volume Reduction Performance

The operation of the Central CSO Regulator has resulted in a considerable volume of combined sewage treated at the wastewater treatment plant that would have otherwise overflowed to the Skagit River. Figure 4-2 provides a scatter plot of monthly wastewater treatment plant flows verses monthly rainfall. The two sets of data points include data from 1988 base flows and the current treatment plant data from December 1997 to June 2000. A linear regression line has been provided for each set of data points. The y-intercept of this graph indicates the base sanitary treated at the plant. The increase of almost 20 mg per month reflects the growth that has occurred in the City over the past 12 years. The slope of the linear regression line reflects the volume of storm water per inch of rainfall that is treated at the wastewater treatment plant. The increase in slope reflects the additional combined sewage that is now being treated at the wastewater treatment plant and additional sources of infiltration and inflow

## Figure 4-2 City of Mount Vernon Monthly Flow vs. Rainfall




Using an average annual rainfall of 32.4 inches, the volume of rain induced flow treated at the plant in 1988 was 199 million gallons (32.4 inches per year x 6.15 million gallons/inch). Currently, the projected rain induced flow treated at the treatment plant is 450 million gallons (32.4 inches per year x 13.88 million gallons/inch). This reflects an increase of 251 million gallons per year. In the City's CSO Reduction Plan the estimated annual overflow volume was 116.5 million gallons. This earlier projection could have been in error or the amount of rain induced flow treated at the plant could have increased significantly. Even if the actual annual overflow volume was only 116.5 million gallons, the Central CSO Regulator has reduced the volume of overflows over 94 percent. This is based on a remaining overflow volume of 6 million gallons per year based on the 6 events identified in Table 4-2.

Using a long term antecedent condition index model the volume fraction of excess flow that is directly attributable to infiltration is up to 70 percent. The infiltration percentage is likely even higher because of the inability to distinguish the infiltration and inflow components based on the information that we have. Based on the flow data that is available at this time it is not possible to identify a unit flow hydrograph distinguishing the three major components of combined sewage flows: sanitary sewage, infiltration, and inflow.

### Central CSO Regulator Solids Reduction Performance

The concentration of TSS in combined sewer overflows has ranged from 14 mg/L to 285 mg/L; however, the treatment plant personnel have documented that the combined sewage generally has concentrations around 50 mg/L. If the annual combined sewer overflow volume was assumed to be 116.5 million gallons with an average concentration of 50 mg/L

volume was assumed to be 116.5 million gallons with an average concentration of 50 mg/L, then the 110.5 million gallon reduction of overflows has reduced the annual total of solids discharged to the Skagit River by 46,000 pounds.

## CSO Reduction Treatment Processes

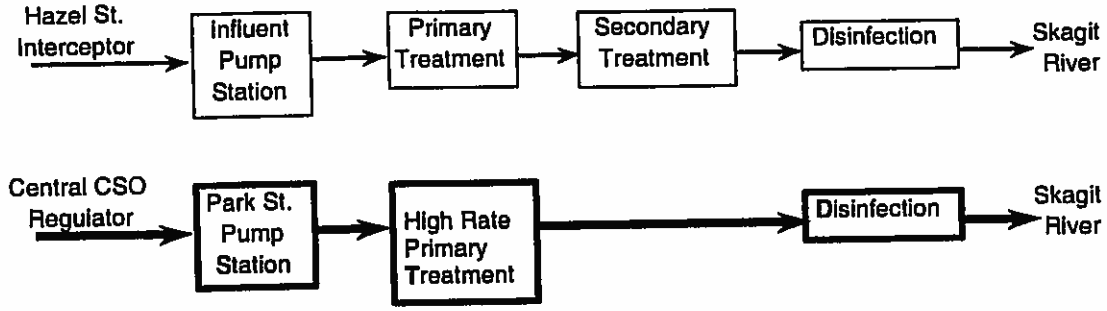
The State of Washington defines CSO treatment as primary treatment that removes at least 50 percent of the total suspended solids (TSS) and an average settlable solids concentration of 0.3 mL/L/hr, with a maximum of 1.9 mL/L/hr. Based on recent CSO treatment projects, the Department of Ecology has interpreted this to be an average annual solids removal requirement.

The 1995 Wastewater Treatment Plant Evaluation identified primary treatment facilities that would be required to reduce overflows to one untreated event per year based in accordance with the City's consent decree. Based on recent CSO treatment projects there are three alternatives for achieving the final reduction requirement in accordance with the consent decree. The primary difference is the level of treatment that is required for the effluent.

### Treatment Alternative 1: CSO Treatment Facility

The first alternative would provide treatment for CSOs similar to the one detailed in the 1995 Wastewater Treatment Plant Evaluation. This treatment alternative would meet the 50 percent removal of the total suspended solids as required by WAC 173-245. A process flow schematic is shown in Figure 4-3. To meet the total peak hour capacity requirements of 48 mgd, high rate primary clarification would be provided. This alternative would require that during CSO events, flows from the Central CSO Regulator would remain separate from the flows through the secondary plant. The CSO treatment would include primary treatment, disinfection and discharge through the outfall. After CSO events the process units could be drained back to the secondary plant.

**Figure 4-3 Alternative 1 CSO Treatment Facility Schematic**



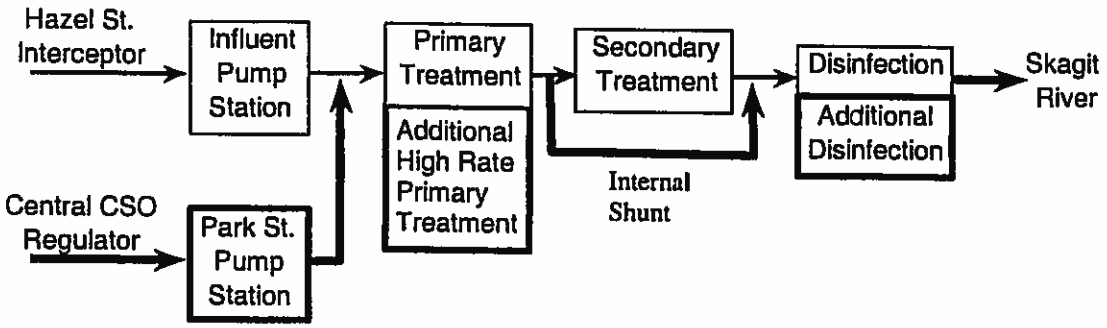

The improvements required for Alternative 1 include:

- Construct conveyance piping from the Park Street Overflow Structure to the Park Street Pump Station.
- Upgrade Park Street Pump Station.
- Construct conveyance piping from Park Street Pump Station to the treatment plant site. This assumes that all or part of the treatment facilities would be located at the secondary treatment plant site.
- Construct primary treatment facilities: Recent experience elsewhere has shown that it is difficult to achieve 50 percent reduction of solids on an event basis with conventional primary treatment when the concentration of the CSO is less than 100 mg/L. High rate clarification using ballasted sedimentation can be used to achieve these requirements. This process could provide greater than 90 percent removal of solids on an event basis.
- Construct dedicated CSO disinfection facilities.
- Construct a CSO outfall dedicated to discharging treated CSOs.

### Treatment Alternative 2: Internal Shunt of CSO Flows, Two Pump Stations

The second alternative would increase the flow rate through the secondary plant. This would require that all discharges meet secondary treatment discharge requirements. To protect the secondary process by preventing 'washout' of the secondary clarifiers during an extreme storm event, the Department of Ecology would likely allow internal shunting of primary effluent directly to the disinfection. Since solids in the Central CSO Regulator are lower than in the Hazel Street Interceptor, it would be preferable to internally shunt the Central CSO Regulator flows. A process flow schematic is shown in Figure 4-4.

**Figure 4-4 Alternative 2 CSO Treatment Internal Shunt Schematic**



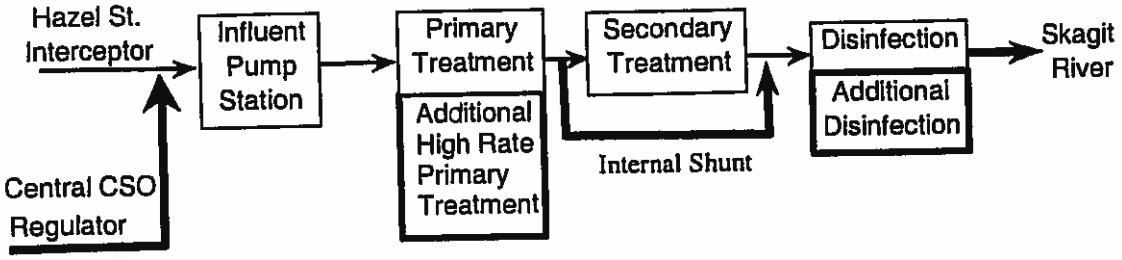

The improvements required for Alternative 2 include:

- Construct conveyance piping from the Park Street Overflow Structure to the Park Street Pump Station.
- Upgrade Park Street Pump Station.
- Construct conveyance piping from Park Street Pump Station to the treatment plant site.
- Construct high rate primary treatment facilities.
- Construct disinfection facilities for both CSO and WWTP flows.
- Construct an outfall to discharging treated CSOs and WWTP effluent. This could be two separate outfalls or a single combined outfall.

### Treatment Alternative 3: Internal Shunt of CSO Flows, One Pump Station

The third alternative would increase the flow rate through the secondary plant, similar to alternative 2 except all of the flows are pumped via the WWTP influent pump station. This would require that all discharges meet secondary treatment discharge requirements. An internal shunt of all CSO flows (from both the Central CSO Regulator and the Hazel Street Interceptor) could occur after initial blending of the flows. A process flow schematic is shown in Figure 4-5.

**Figure 4-5 Alternative 3 CSO Treatment Internal Shunt Schematic**




The improvements required for Alternative 3 include:

- Construct a new influent pump station.
- Construct conveyance piping from Park Street Overflow Structure.
- Construct high rate primary treatment facilities.
- Construct disinfection facilities for both CSO and WWTP flows.
- Construct an outfall to discharging treated CSOs and WWTP effluent. This could be two separate outfalls or a single combined outfall.

### Summary of Treatment Alternatives

Table 4-3 presents a summary of the treatment requirements for each Alternative, and the improvements required.

Table 4-3

| Summary of CSO Treatment Alternatives | | |
|---|---|---|
| **Alternative No. 1** | **Alternative No. 2** | **Alternative No. 3** |
| **Description** | | |
| CSO Treatment Facility | Internal Shunt of CSO Flows, Two Pump Stations | Internal Shunt of CSO Flows, One Pump Station |
| **Treatment Requirements** | | |
| 50 Percent Solids Removal, 0.3 mL/L/hr settlable solids (max of 1.9 mL/L/hr)[1] | NPDES Permit Limits:<br>30 mg/L BOD and TSS | NPDES Permit Limits:<br>30 mg/L BOD and TSS |
| **Required Improvements** | | |
| High Rate Primary Treatment for CSO flows | High Rate Primary Treatment for CSO flows | High Rate Primary Treatment for CSO flows |
| Disinfection for CSO flows | Disinfection for CSO flows | Disinfection for CSO flows |
| Upgrade Influent Pump Station | Upgrade Influent Pump Station | Construct new Influent Pump Station |
| Upgrade the Park Street Pump Station and replace piping to Park Street Pump Station | Replace piping to Park Street Pump Station | Upgrade Hazel Street Interceptor CSO Regulator to Influent Pump Station |
| Provide dedicated CSO Outfall | Provide an additional outfall capacity | Upgrade WWTP Outfall or provide an additional outfall capacity |
| Forcemain from Park Street Pump Station to WWTP | Force Main from Park Street Pump Station to WWTP | |

1. Based on NPDES Permit issued to Carkeek CSO Treatment Facility, King County, WA.

## CSO Storage

Both in-line, such as the Central CSO Regulator, and off-line storage facilities were considered for the remaining CSO flows. For storage facilities, a large factor of safety should be incorporated to allow the facility to accommodate both short duration high intensity storms and long duration low intensity storms, which may activate all sources of inflow and infiltration. From an idealized hydrograph with a peak of 48 mgd, as previously shown in Figure 3-5, a minimum of 1.0 mg of excess volume would be required to be stored. For CSOs, rainfall patterns, impervious area within the city, and antecedent moisture conditions can affect the actual volume experienced. Because of the variability in

these factors, the CSO volume that would be planned for would incorporate a safety factor of 2.0. A CSO storage alternative would require a 2.0 mg storage facility at an estimated cost of $12.0 million

## CSO Separation

A portion of the CSO flows in the Mount Vernon sewer system is from inflow sources, such as direct connections of storm drain catch basins. Identification and separation of inflow sources could reduce or eliminate the need for additional storage or CSO treatment facilities. However, identification and removal of direct connections is not always possible. In addition, in many cases the excess flows experienced due to a storm event are from rapid infiltration sources, rather than inflow sources, which are difficult to identify and correct. In the case of Mount Vernon, if excess CSO flows were due to inflow, an area of 136 acres would be connected. In the one-year storm event, inflow is typically due to run-off from paved areas, streets and parking lots that drain to the CSO system. To remove 136 acres of impervious area would require approximately 37 miles of 30 foot wide streets to be identified and disconnected.

## Recommended CSO Reduction Treatment Alternative

### Treatment Criteria

The two treatment options, separate CSO treatment and internally shunted flows, have different effluent requirements:

- The performance goal of CSO Treatment is removal of 50 percent suspended solids on an annual basis. Additionally, the effluent settleable solids concentration must have an annual average of 0.3 mL/L/hr, with a maximum of 1.9 mL/L/hr.

- Internal Shunt is the name given to the treatment of CSO flows by primary treatment followed by blending with secondary treatment plant flows before disinfection. When flows are internally shunted, the blended effluent from the primary and secondary units must meet the weekly and monthly NPDES permit (BOD and TSS) limits. DOE has permitted this process at other plants in the northwest including King County's West Point Treatment Plant.

### Treatment Recommendation

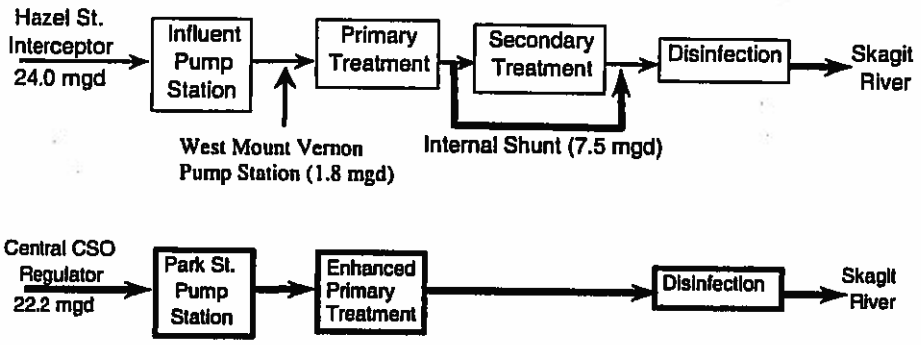
To meet the total 48 mgd peak hour flow requirements, treatment of CSO flows would be performed similar under either alternative. Primary treatment of CSO flows could be via high rate clarification and disinfection by UV. The typical operating cost of treating all flows via an internal shunt and treating them via a CSO treatment facility is similar. Similar improvements (additional primary treatment process equipment and UV disinfection equipment) are required for all Alternatives. Costs for Alternative no. 3 would be far in

excess of either Alternative nos. 1 or 2 since it requires a new pump station (the influent pump station would need to be replaced rather than upgraded) and upgrading the Hazel Street Interceptor. Alternative nos. 1 and 2 would be similar in cost, so the decision of treatment Alternative (internal shunt vs. CSO treatment facility) should be based on treatment requirements.

The Hazel Street Interceptor conveys both combined and sanitary flows to the influent pump station. This interceptor has a capacity of 24.0 mgd. It is recommended that the portion of flows in excess of the peak sanitary flows (18.3 mgd) be internally shunted. This will allow maximization of the WWTP, without the necessity of oversizing all process units to accommodate CSO flows. Furthermore, by internally shunting CSO flows, the blended effluent may be able meet the NPDES permit requirements.

The Central CSO Regulator has lower TSS and BOD than the Hazel Street Interceptor since it conveys only combined sewer flows. It is recommended that wastewater conveyed by the Central CSO Regulator be treated in an independent treatment process. The treatment requirements for this process will be based on CSO treatment requirements (50 percent solids removal on an average annual basis, with an average settleable solids of 0.3 mL/L/hr, and a maximum of 1.9 mL/L/hr). The process flow diagram for these recommendations is presented in Figure 4-6.

**Figure 4-6 Recommended Process Schematic Flow Diagram**




## CSO Reduction Alternatives

The treatment Alternative recommended for the combined flows is composed of two components: An 'Internal Shunt' and CSO Treatment. The 'Internal Shunt' of the Hazel Street Interceptor is discussed in Chapters 7 to 10 under the Wastewater Treatment Plant. Three alternatives for CSO Treatment are presented below.

### Alternative 2A: Treat and Disinfect Combined Wastewater at the Park Street Pump Station.

Alternative 2A consists of treatment (high rate clarification) and disinfection (UV) at the Park Street Pump Station location. Improvements required for this alternative include:

- Construct a high rate clarification unit;
- Construct a UV disinfection system;
- Construct a 36-inch diameter sewer from the Park Street Overflow Structure to the Park Street Pump Station;
- Upgrade Park Street Pump Station to separate and convey CSO and storm flows;
- Construct a CSO effluent pump station; and
- Construct an outfall for this CSO treatment facility effluent.

The estimated capital cost of this alternative is $9.2 million.

**Alternative 2B: Treat Combined Wastewater at the Park Street Pump Station and Disinfect at the WWTP.**

Alternative 2B consists of treatment (high rate clarification) at the Park Street Pump Station and disinfection (UV) at the WWTP. Improvements required for this alternative include:

- Construct a high rate clarification unit at the Park Street Pump Station;
- Retrofit a UV disinfection system in the existing chlorine contact basin at the WWTP;
- Construct a 36-inch diameter sewer from the Park Street Overflow Structure to the Park Street Pump Station;
- Upgrade Park Street Pump Station to separate and convey CSO and storm flows;
- Construct a forcemain from Park Street Pump Station to the WWTP;
- Retrofit a CSO effluent pump station in the existing chlorine contact basin; and
- Construct conveyance to the outfall for treated CSO effluent.

The estimated capital cost of this alternative is $9.9 million.

**Alternative 2C: Treat and Disinfect Combined Wastewater at the WWTP.**

Alternative 2C consists of treatment (high rate clarification) and disinfection (UV) at the WWTP. Improvements required for this alternative include:

- Construct a high rate clarification unit at the WWTP;

- Retrofit a UV disinfection system in the existing chlorine contact basin at the WWTP;
- Construct a 36-inch diameter sewer from the Park Street Overflow Structure to the Park Street Pump Station;
- Upgrade Park Street Pump Station to separate and convey CSO and storm flows;
- Construct a forcemain from Park Street Pump Station to the WWTP;
- Retrofit a CSO effluent pump station in the existing chlorine contact basin; and
- Construct conveyance to the outfall for treated CSO effluent.

The estimated capital cost of this alternative is $9.6 million.

### Comparison of Alternatives

The ***benefits*** of alternative 2A are as follows:

- Capital cost of the project is estimated to be approximately $400,000 less than the other alternatives.

The ***disadvantages*** of alternative 2A are as follows:

- Remote location (away from the WWTP) requiring additional time or staff to maintain and operate;
- Requires construction of a dedicated CSO outfall;
- Limited ability to utilize process equipment for alternate uses (such as WWTP redundancy or effluent polishing during non-CSO periods); and
- Requires permitting and new construction in the flood way, which may be difficult to obtain.

The advantages of alternative 2B are as follows:

- Ability to utilize the UV Disinfection process equipment for redundancy, and during maintenance or repair of the WWTP's UV system; and
- Ability to utilize the WWTP outfall for disposal of treated CSO flows.

The disadvantages of alternative 2B are as follows:

- Remote location (away from the WWTP) of the high rate clarification requires additional time or staff to maintain and operate; and

- Capital cost of the project is estimated to be approximately $400,000 more than alternative 2A.

- Requires permitting and new construction in the flood way, which may be difficult to obtain.

The advantages of alternative 2C are as follows:

- Ability to utilize the UV Disinfection process equipment for redundancy, and during maintenance or repair of the WWTP's UV system; and

- Ability to utilize the WWTP outfall for disposal of treated CSO flows.

The disadvantages of alternative 2C are as follows:

- Capital cost of the project is estimated to be approximately $400,000 more than alternative 2A.

### RECOMMENDED CSO REDUCTION ALTERNATIVE

Alternative 2C is the recommended treatment facility alternative. The differential in cost is easily offset by the potential to utilize both the high rate clarification and UV Disinfection systems as redundant unit processes for the WWTP during non-storm event periods. Table 4-4 summarizes the recommended CSO Reduction Plan.

Table 4-4

| Summary of CSO Reduction Plan Improvements | | |
|---|---|---|
| **CSO Reduction Method** | **Description/Benefit** | **Required Improvements** |
| Phase 1 | Central CSO Regulator provides inline storage of CSO flows that would have been conveyed to the Skagit River. Stored CSO flows are conveyed to the WWTP as capacity allows for treatment and disposal. | In-line storage. Completed and online, December 1997 |
| Phase 2[1] | The 'Internal Shunt' of Hazel Street Interceptor CSO Flows would allow a peak flow of approximately 7.5 mgd to be continually treated during a storm event. This additional treatment capacity will allow the CSO regulator to act as equalizing in-line storage and further reduce the potential CSO events. | Increase capacity of the influent pump station.<br><br>Increase capacity of the headworks, primary treatment facilities, disinfection system, effluent pump station, and secondary WWTP outfall for a hydraulic capacity of 25.8 mgd.<br><br>Add the potential for coagulant addition to the primary clarifier designated for CSO treatment. |
| Phase 3 | The CSO Treatment Facility will be final phase of CSO reduction. It will allow the City meet their consent decree with DOE and reduce CSOs to less than one untreated event per year. | Construct a high rate clarification system with a peak hour capacity of 22.2 mgd.<br><br>Construct a UV disinfection system<br><br>Construct a 750 LF of 36-inch sewer.<br><br>Upgrade Park Street Pump Station<br><br>Construct 1500 LF of 30-inch force main<br><br>Construct a CSO effluent pump station<br><br>Construct conveyance to the secondary effluent outfall[2] |

1. Improvements for Phase 2, the Internal Shunt of CSO flows, are included in Chapter 10, Recommended Alternatives.
2. It is assumed that treated CSO flows and the secondary effluent will be combined and discharged through the same outfall.

Phase 3 of the CSO reduction plan is the construction of a CSO treatment facility to reduce untreated CSOs to less than one event per year. A CSO treatment facility is assumed be subject to the following treatment requirements (based on the NPDES discharge permit issued to the Carkeek CSO Treatment Facility, King County, WA):

- Removal of 50 percent suspended solids on an annual basis;
- An annual average of effluent settleable solids concentration of 0.3 mL/L/hr; and
- A maximum effluent settleable solids concentration of 1.9 mL/L/hr.

The key treatment processes of a CSO treatment facility would include high rate clarification for removal of suspended solids, and UV disinfection for disinfection of effluent:

- High rate clarification (HRC) is a physical/chemical process that utilizes high specific gravity ballast material, such as sand, to increase the settling velocities of particulate matter or chemically conditioned floc particles. The benefits of HRC is that it requires a small footprint, has a rapid start-up time, and produces an effluent low in turbidity and suspended solids.
- UV disinfection is the process whereby wastewater is exposed to UV energy which, when absorbed by micro-organisms, damages the nucleic acid preventing reproduction of the organism and eliminating the ability of the micro-organism to cause infections. UV disinfection has benefits over chlorine for CSO applications in that it does not degrade over time, does not require large volume of chlorine to be stored on site, and does not require large contact tanks to be constructed.

The estimated costs for a CSO Treatment Facility for the City of Mount Vernon are presented in Table 4-5. These costs include conveyance of CSOs to the wastewater treatment plant site, construction of CSO treatment facilities, treated CSO disposal, and an estimate of the annual operations and maintenance costs of the CSO Treatment Facility.

Table 4-5

| Recommended Improvements for the CSO Treatment Facility[1,2] | |
|---|---|
| **Improvement** | **Capital Cost Estimate ($1,000)** |
| CSO Interceptor | $700 |
| Upgrade Park Street Pump Station | $700 |
| CSO Forcemain | $500 |
| CSO Treatment (High Rate Clarification) | $4,200 |
| CSO Disinfection[3] | $2,200 |
| CSO Effluent Pump Station | $800 |
| CSO Outfall[4] | --[4] |
| Total Capital Cost | $9,100 |
| Estimated Annual O & M Cost[5] | $8.4 to $9.6 |

1. Does not include CSO-Phase 2 Improvements, which are incorporated in secondary treatment plant improvements - presented in Chapter 10.
2. CSO Phase 3 Improvements, per DOE Consent Decree, may be required by 2015
3. Based on an estimated transmissivity of the treated CSO effluent.
4. Costs are included in the WWTP single outfall estimate, where both treated CSO flows and secondary effluent are discharged through a single outfall.
5. Based on the average overflow volume for 1998-2000 (5.6 mg), and a cost estimate of $1.50 to $1.75 per 1,000 gallons treated.

# 5. WASTEWATER COLLECTION SYSTEM

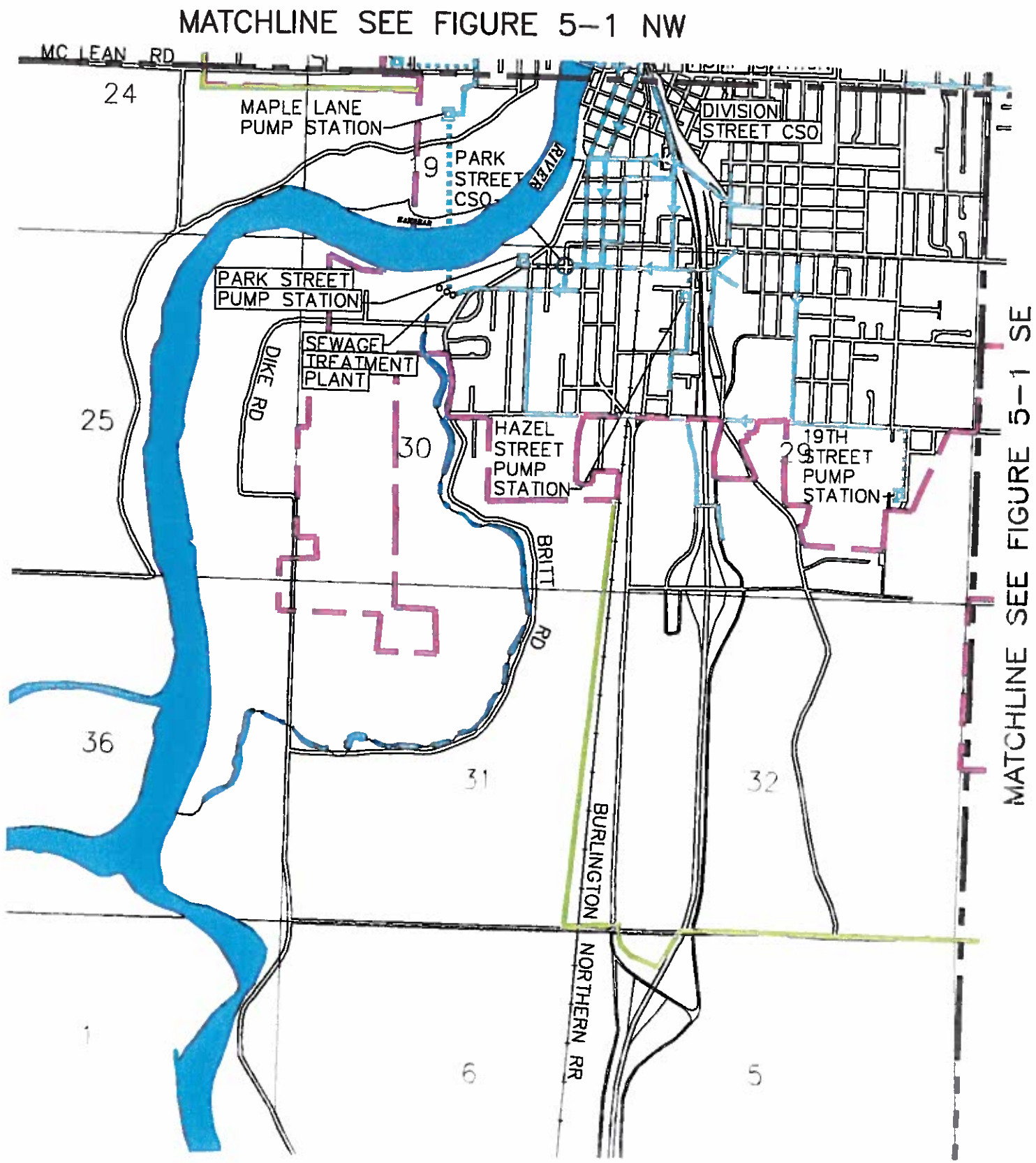
This chapter presents an evaluation of the wastewater collection system. It includes a review of the interceptor system capacity based on projected peak flows, using the population projections presented in Chapter 3. A review of the City's Access database of sewer defects was also completed. The following sections identify system deficiencies, summarize corrective actions and costs required to correct the defects, and future improvements to the interceptor system required for projected growth.

## SYSTEM DESCRIPTION

The City of Mount Vernon's wastewater collection system presently serves an area of approximately sixteen square miles. Figure 5-1 shows the major sewer lines, pump stations, and combined sewer overflow structures in the system. The system is composed of approximately 120 miles of pipe ranging from 6-inch to 60-inch diameter. The majority of the wastewater collection system was constructed of concrete pipe. The system pipe materials also include clay, corrugated metal, PVC, and polyethylene.

Portions of the downtown and older areas are served by combined sewers. Separate storm sewers are provided in the newer developed areas. The wastewater collection system was reviewed in 1994 and deficiencies in the system identified. Each year the City has allocated monies to repair known deficiencies.

The wastewater collection system presently includes thirteen pump stations owned and operated by the City, Table 2-1. The City also maintains and operates three combined sewer overflow structures (Freeway Drive, Division Street, and Park Street) and two CSO/storm water pump stations (Division Street and Park Street), see Chapter 4.




LEGEND:

- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROW)
- EXISTING FORCE MAIN
- EXISTING PUMP STATION
- EXISTING OVERFLOW WEIR
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc.

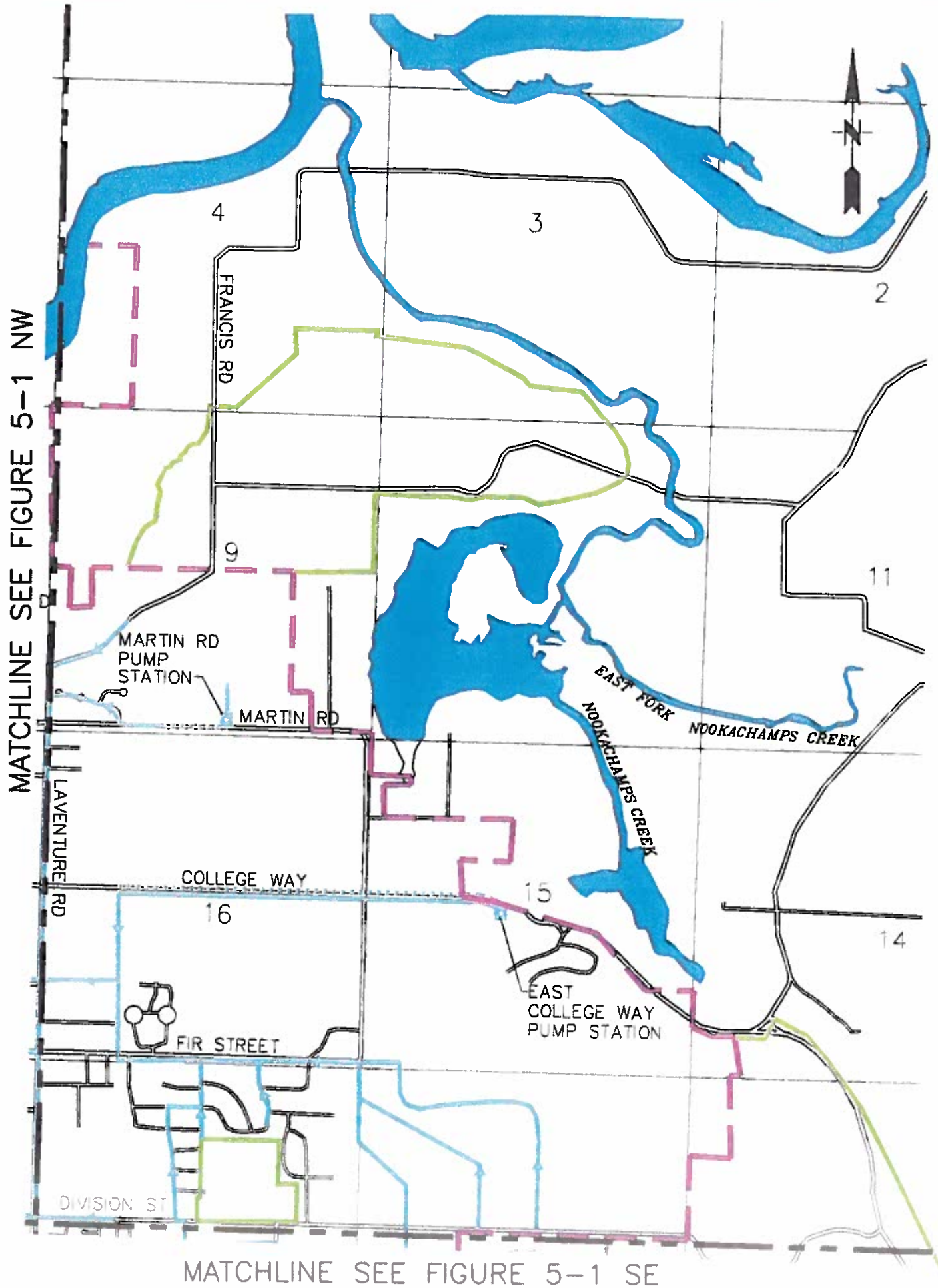
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No.: 5-1 SW

DATE: 02/12/03




LEGEND:

- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROW)
- EXISTING FORCE MAIN
- EXISTING PUMP STATION
- EXISTING OVERFLOW WEIR
- CITY LIMITS
- UGA BOUNDRY

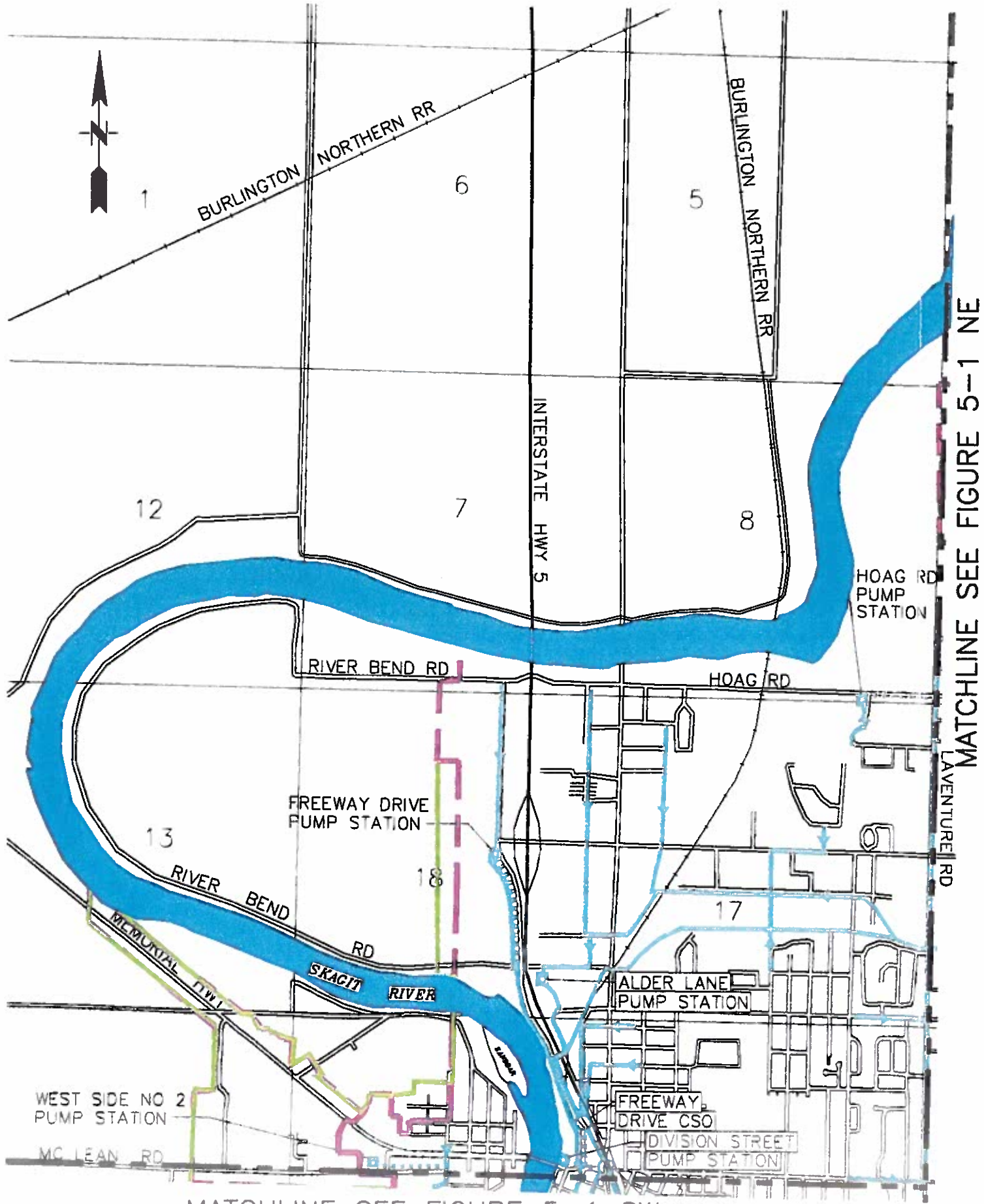
HDR Engineering, Inc.

Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No.: 5-1 NE




LEGEND:

- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROW)
- EXISTING FORCE MAIN
- EXISTING PUMP STATION
- EXISTING OVERFLOW WEIR
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc

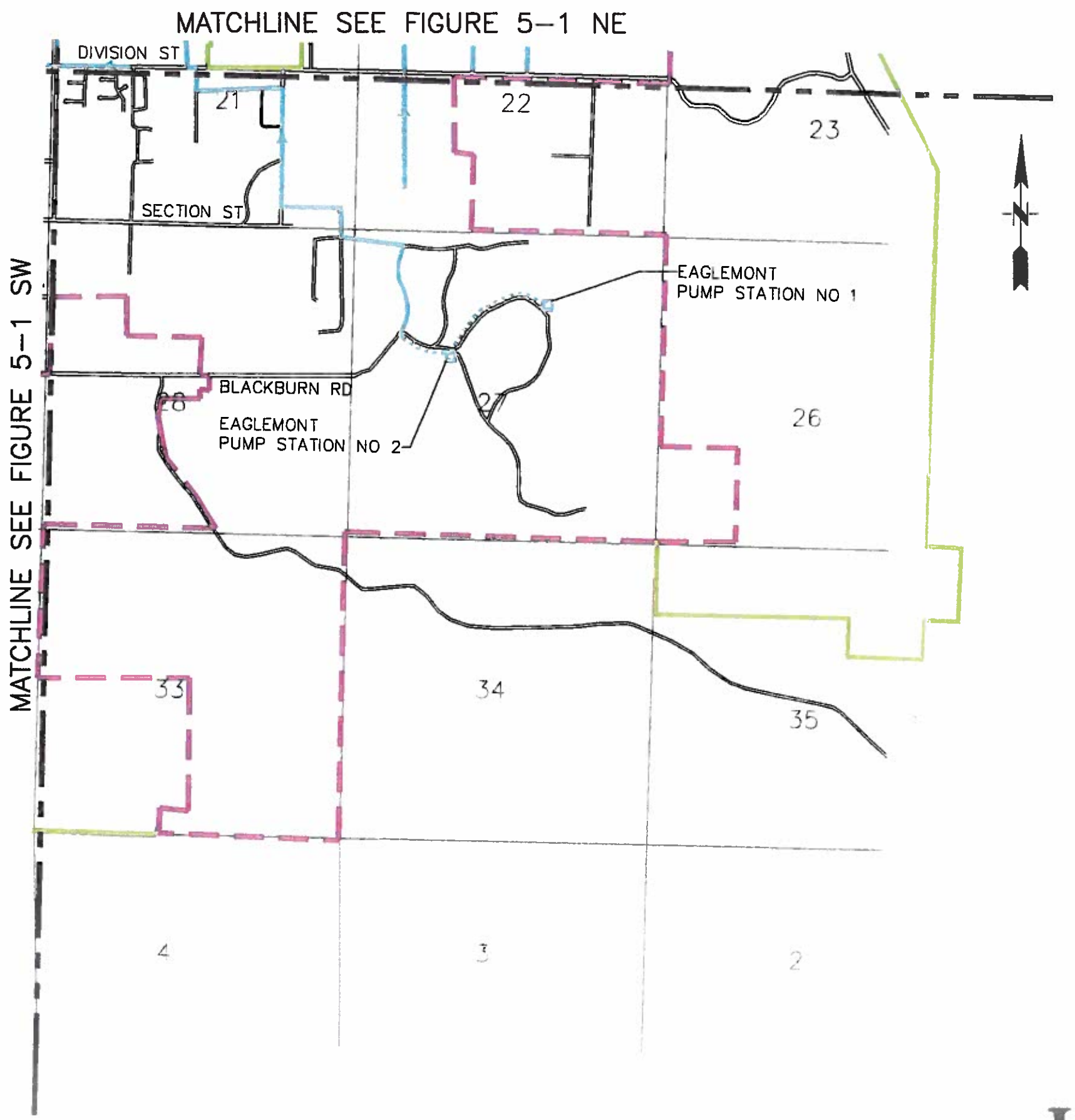
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No: 5-1 NW

DATE: 02/12/03




LEGEND:

- EXISTING MAIN COLL SEWERS (W/ FLOW
- EXISTING FORCE MAI
- EXISTING PUMP STA
- EXISTING OVERFLOW
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc

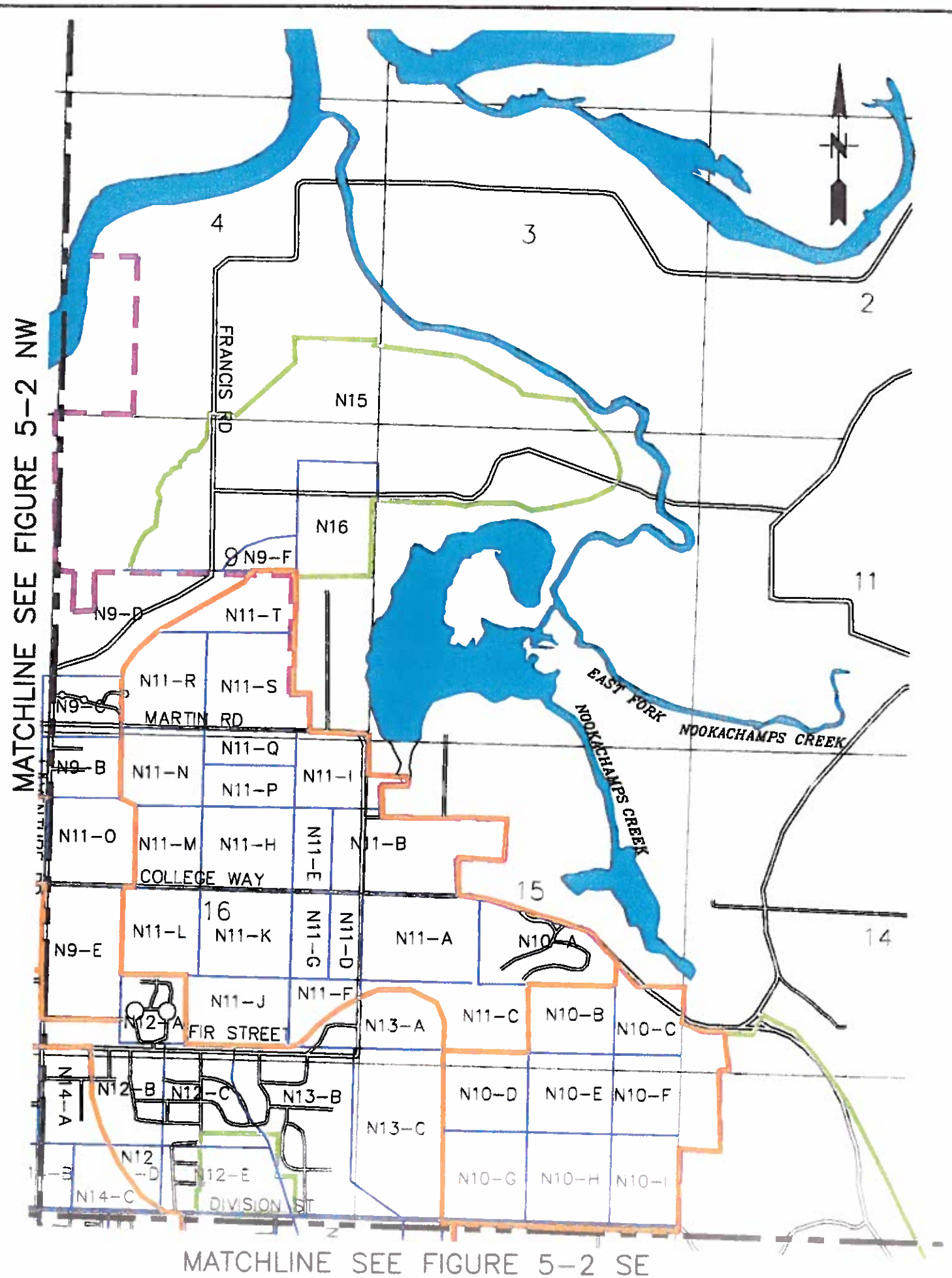
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No: 5-1 SE

DATE: 02/12/03




## WASTEWATER COLLECTION SYSTEM CAPACITY ASSESSMENT

### Introduction

An analysis of the capacity of existing interceptors and major trunk lines was completed to determine hydraulic limitations within the system that could limit future development. Figure 5-1, presented previously, provides the location of the existing wastewater collection system interceptors. Wastewater flows were developed in Chapter 3, and are based on Skagit County Population Projections for the 20-year planning horizon, through 2020.

### Analysis

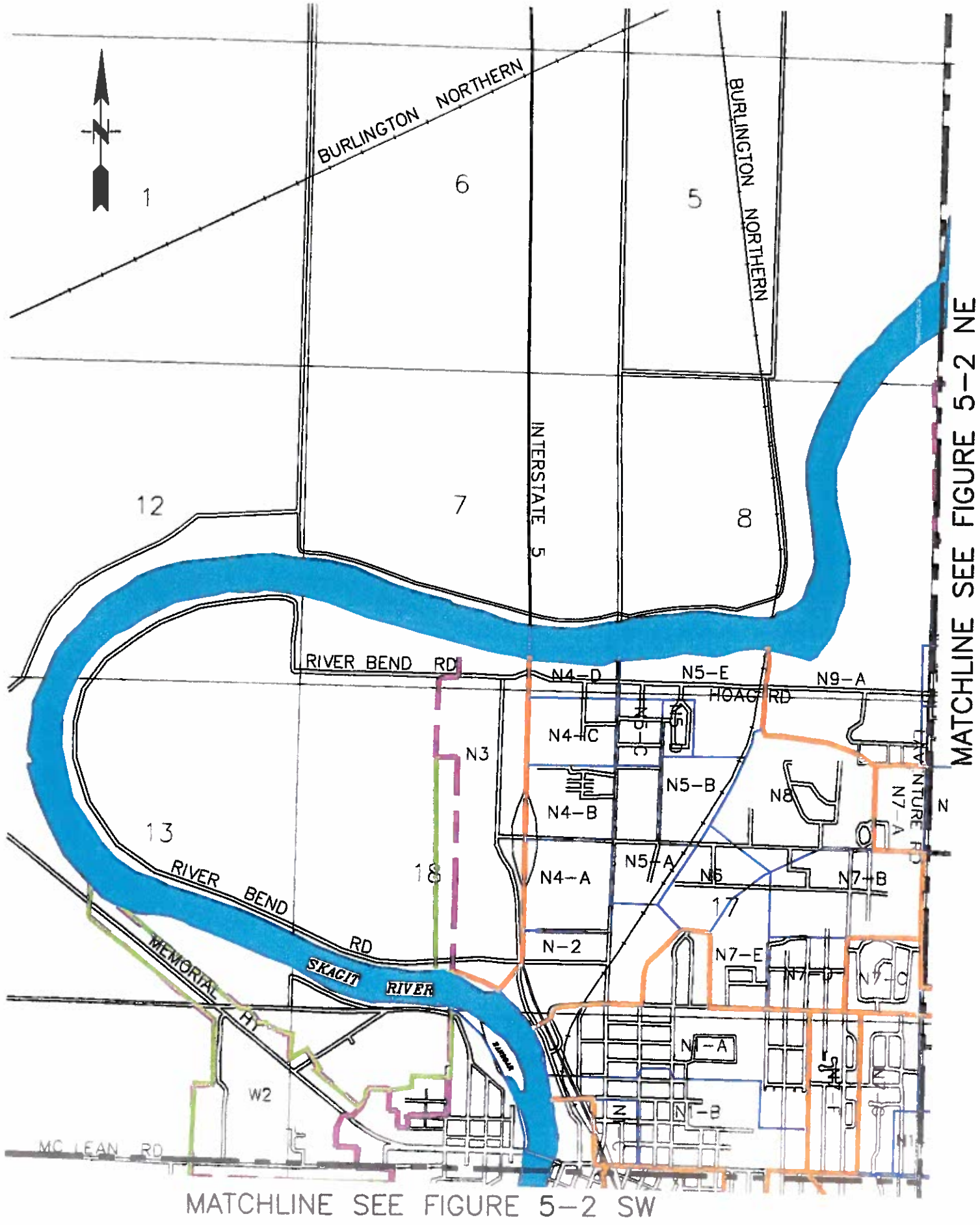
The system analysis was completed by defining the interceptors and major trunk lines. Manhole invert elevations and pipe lengths between manholes in the defined segments were obtained from City utility mapping and previous HYDRA modeling efforts. The analysis was completed by developing flow components for a fully developed UGA for each drainage area, Figure 5-2. Area, population density, and flow contribution assigned to each drainage sub-basin are presented in Appendix C. Flows from each drainage basin were estimated, including infiltration and inflow and peak sanitary flows, based on the following parameters:

| | |
|---|---|
| Average Daily Per Capita Flow | 100 gpcd |
| Infiltration and Inflow Rates | 1,100 gpad |
| Peaking Factor for Sanitary Flows | L.A. Peaking Curve[1] |

1. Fig. 3-6 Ratio of Peak Flow to Average Daily Flow in Los Angeles, ASCE Manual and Report on Engineering Practice No. 60.

The hydraulic capacity of each line segment was determined and compared to the future flows in the pipe. A sample analysis is presented in Appendix D.

In general, the interceptor system has few lines that have or will approach their capacity at full development. Flow monitoring, additional study, and modeling of the interceptors in the northern portions of the collection system would allow a more accurate prediction of when the new interceptors are required. Table 5-1 lists the lines identified by the hydraulic analysis as having limited capacity given the growth projections and the current UGA boundary.




LEGEND

- CITY LIMITS
- UGA BOUNDRY
- SUB-BASIN BOUNDRY
- DRAINAGE BASIN BOUNDRY

HDR Engineering, Inc

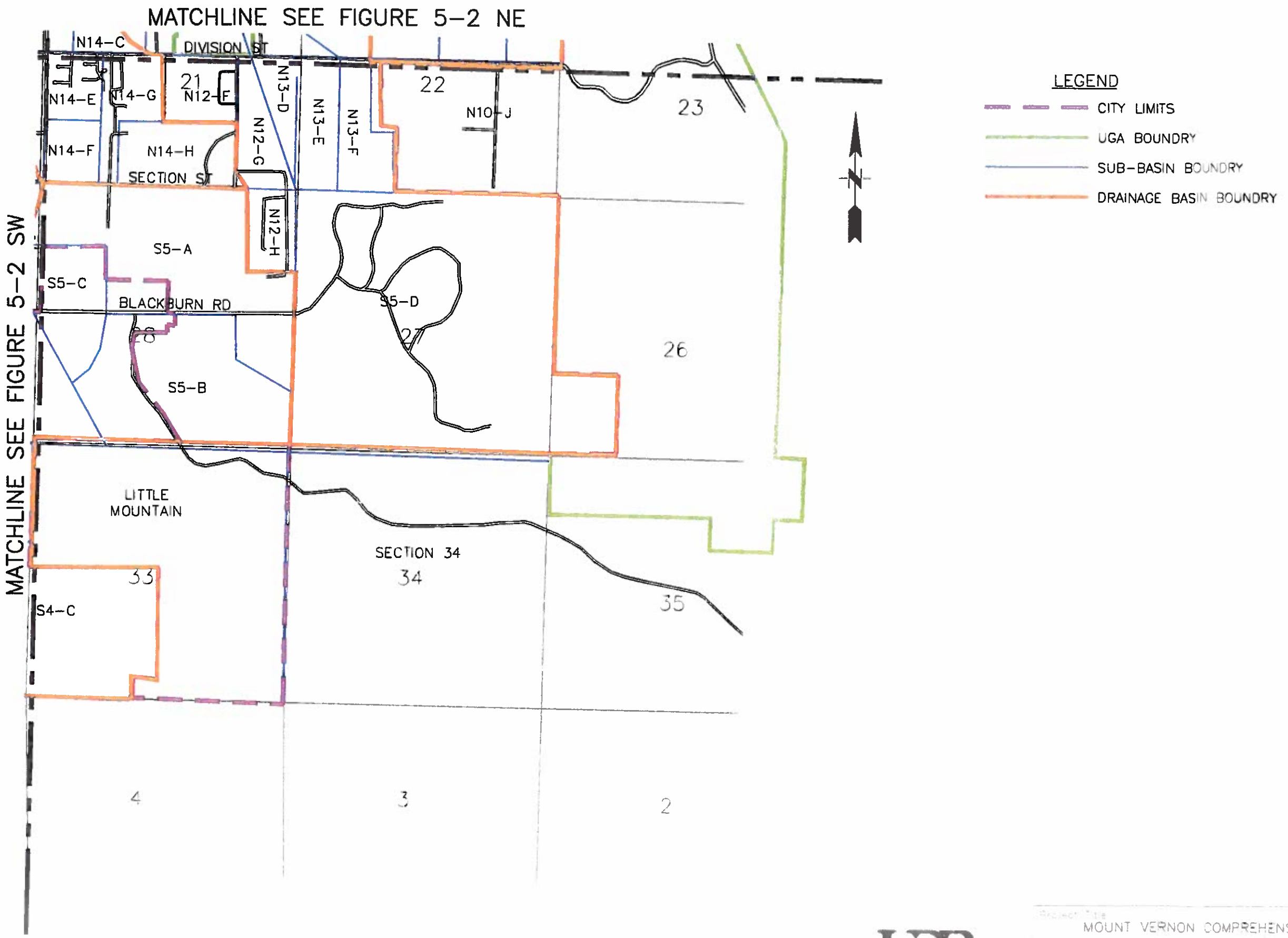
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: DRAINAGE BASINS

Figure No: 5-2 NW

DATE: 02/12/03




LEGEND

- CITY LIMITS
- UGA BOUNDRY
- SUB-BASIN BOUNDRY
- DRAINAGE BASIN BOUNDRY

HDR Engineering, Inc.

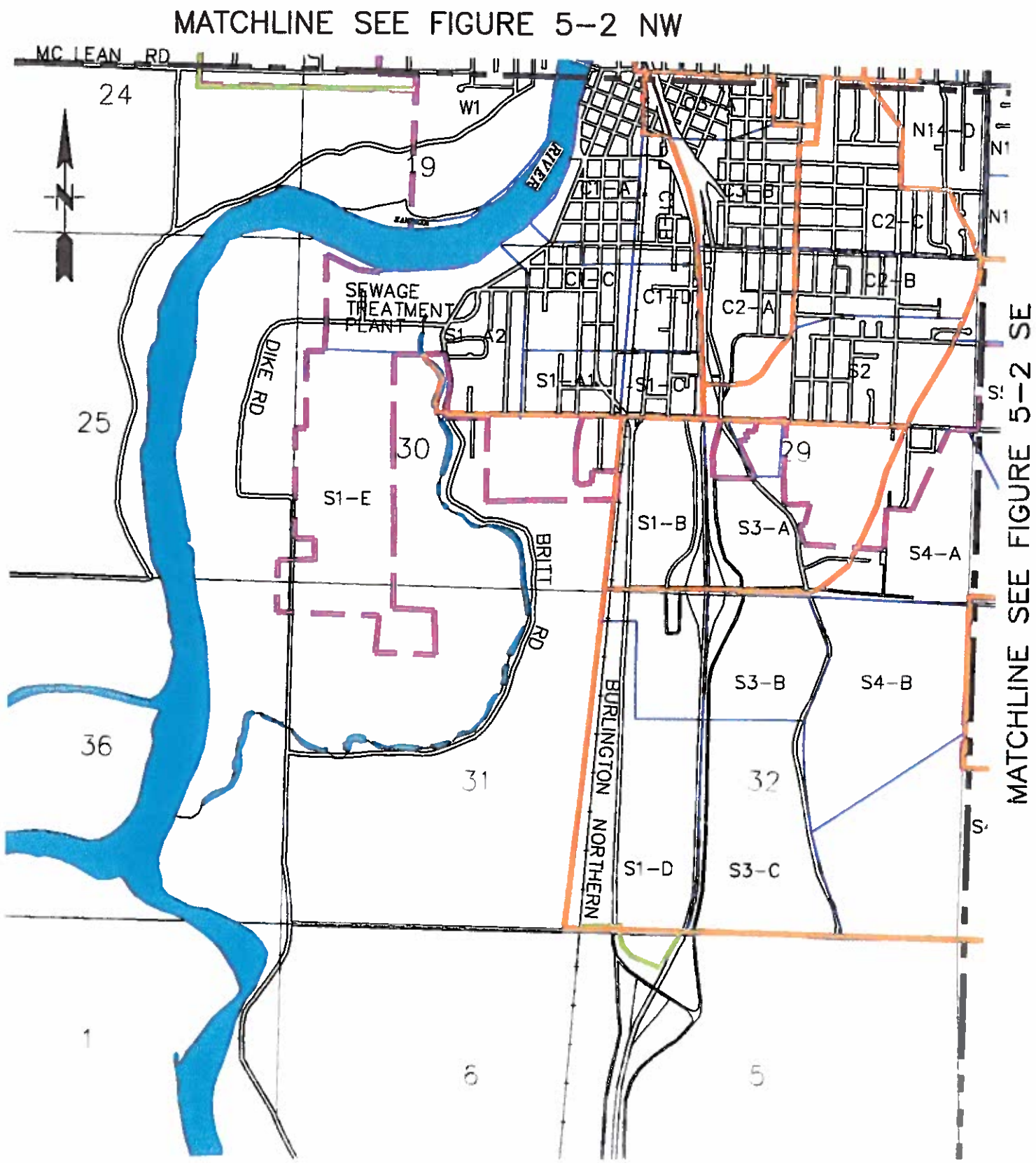
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: DRAINAGE BASINS

Figure No.: 5-2 SE

DATE: 02/12/03




LEGEND

- CITY LIMITS
- UGA BOUNDRY
- SUB-BASIN BOUNDRY
- DRAINAGE BASIN BOUNDRY

HDR Engineering, Inc

Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: DRAINAGE BASINS

Figure No: 5-2 SW

DATE: 02/12/03

## WASTEWATER COLLECTION SYSTEM CAPACITY ASSESSMENT

### Introduction

An analysis of the capacity of existing interceptors and major trunk lines was completed to determine hydraulic limitations within the system that could limit future development. Figure 5-1, presented previously, provides the location of the existing wastewater collection system interceptors. Wastewater flows were developed in Chapter 3, and are based on Skagit County Population Projections for the 20-year planning horizon, through 2020.

### Analysis

The system analysis was completed by defining the interceptors and major trunk lines. Manhole invert elevations and pipe lengths between manholes in the defined segments were obtained from City utility mapping and previous HYDRA modeling efforts. The analysis was completed by developing flow components for a fully developed UGA for each drainage area, Figure 5-2. Area, population density, and flow contribution assigned to each drainage sub-basin are presented in Appendix C. Flows from each drainage basin were estimated, including infiltration and inflow and peak sanitary flows, based on the following parameters:

| | |
|---|---|
| Average Daily Per Capita Flow | 100 gpcd |
| Infiltration and Inflow Rates | 1,100 gpad |
| Peaking Factor for Sanitary Flows | L.A. Peaking Curve[1] |

1. Fig. 3-6 Ratio of Peak Flow to Average Daily Flow in Los Angeles, ASCE Manual and Report on Engineering Practice No. 60.

The hydraulic capacity of each line segment was determined and compared to the future flows in the pipe. A sample analysis is presented in Appendix D.

In general, the interceptor system has few lines that have or will approach their capacity at full development. Flow monitoring, additional study, and modeling of the interceptors in the northern portions of the collection system would allow a more accurate prediction of when the new interceptors are required. Table 5-1 lists the lines identified by the hydraulic analysis as having limited capacity given the growth projections and the current UGA boundary.

Table 5-1

| Hydraulic Analysis Identified Capacity Limitations at Saturated Development[1] | | | |
|---|---|---|---|
| **Location** | **between** | **Comment** | **Interceptor/ Trunk Sewer** |
| East of City Limits (sections 23 and 26) | | Parallel line to College Way Pump Station | Future |
| East of City Limits (sections 15 and 22) | | Parallel line to College Way Pump Station | Future |
| Martin Road | Trumputer and College Way | Monitor existing 8-inch | College Way |
| College Way | Martin Road and 35th St | Monitor existing 12-inch | College Way |
| College Way | Martin Rd to College Way Pump Station | Replace existing 8-inch | College Way |
| Fir Street | 30th St and Comanche Dr | Monitor existing 12-inch | Fir Street |
| Fir Street | 30th Street and 26th Street | Monitor existing 12-inch | Fir Street |
| 26th Street | Jacqueline and Kulshan | Monitor existing 12-inch | Fir Street |
| 26th Street | College Way and Kulshan Avenue | Reroute flows from College Way Pump Station[2] | Fir Street |
| LaVenture Road | Division Street and Fir Street | Monitor existing 8-inch | LaVenture |
| LaVenture Road | Fir St and Kulshan Ave | Replace existing 8-inch | LaVenture |
| LaVenture Road | Fir St and Kulshan Ave | Replace existing 10-inch | LaVenture |
| Kulshan Interceptor | Minimal slope: 24- and 30-inch pipe | Designed to operate under surcharged conditions. | Kulshan |
| Burlington Northern Railroad | South of Roosevelt Ave | Replace existing 15-inch | Alder Lane |

| Hydraulic Analysis Identified Capacity Limitations at Saturated Development[1] | | | |
|---|---|---|---|
| **Location** | **between** | **Comment** | **Interceptor/ Trunk Sewer** |
| Blackburn Road | East of Walter St | Monitor existing 30-inch | Southeast |
| Walter Street | Blackburn Rd and Hazel St | Monitor existing 30-inch | Southeast |
| Urban Avenue | North of College Way | Monitor existing 10-inch | Urban Ave |
| Freeway Drive | River Bend Rd and Cameron Way | Monitor existing 8- and 10-inch | Freeway Dr |

1. Based on saturated development within the current GMA at present zoning.
2. Rerouted flows include construction of a forcemain, gravity mains, and upgrading the College Way Pump Station.

### Interceptor System Improvements

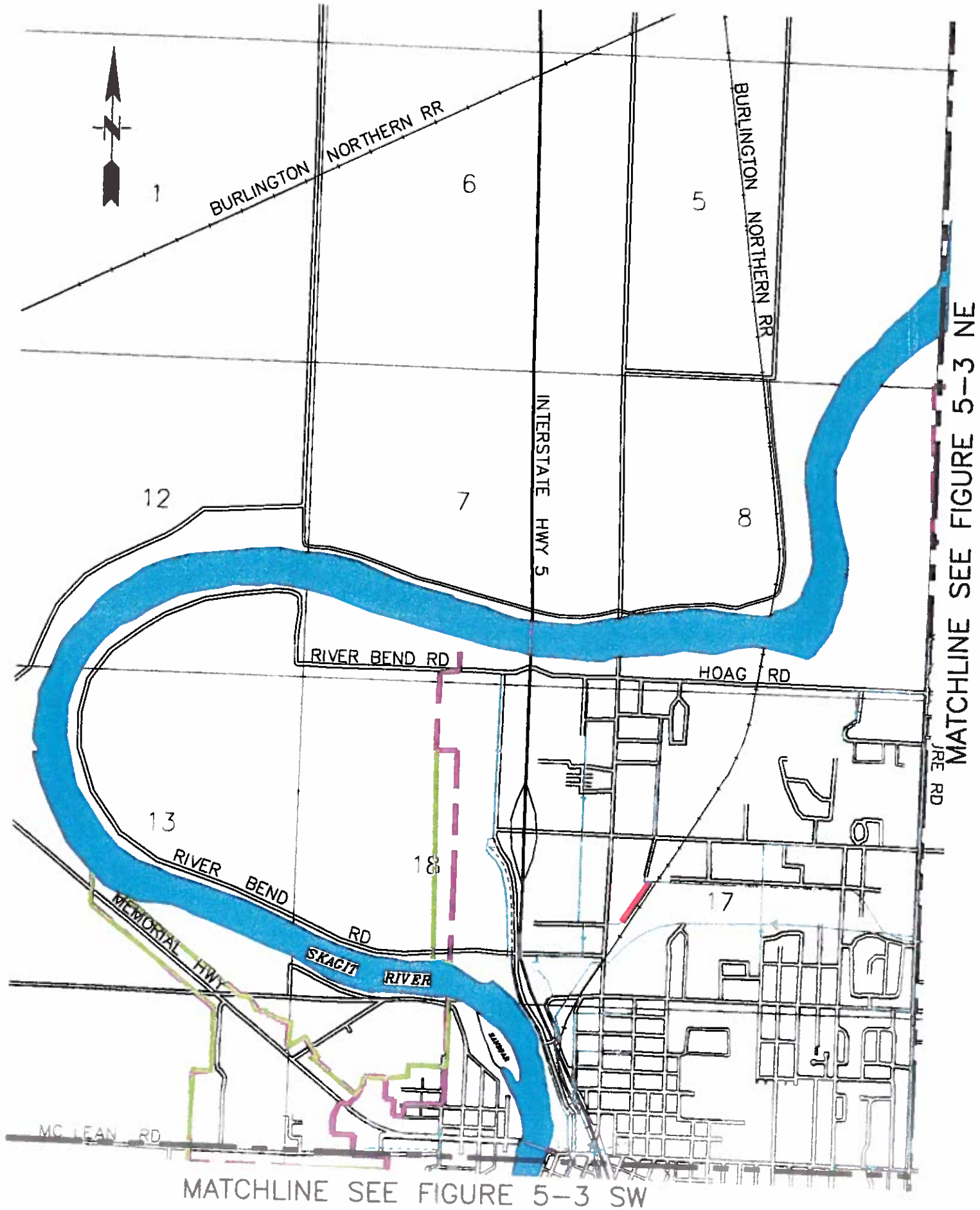
The interceptor system has lines that are predicted to approach capacity as the UGA approaches saturated development. These lines are recommended for monitoring and replacement as warranted. The following sections provide details of each of the interceptors. Table 5-2 summarizes the recommendations for each identified defect. Figure 5-3 presents the improvements to the interceptors and trunk sewer system based on the hydraulic analysis.

**Table 5-2**

| Interceptor System Improvements | | | | | | |
|---|---|---|---|---|---|---|
| **ID No.** | **Location** | **between** | **Year Required** | **Dia (in)[1]** | **Length (ft)[1]** | **Cost ($1,000)[2]** |
| FS-1 | Sections 23 and 26 | | Future | 18 | 1,379 | 380 |
| FS-2 | Sections 15 and 22 | | Future | 18 | 1,063 | 295 |
| FS-3 | Martin Rd | Trumpter Rd. and College Way | As-Required | 12 | 734 | 135 |
| FS-4 | College Way | Martin Rd. and 35th St. | As-Required | 15 | 548 | 125 |
| FS-5 | College Way | Martin Rd. to Pump Station | 2002 | 18 | 2,307 | 635 |

| Interceptor System Improvements | | | | | | |
|---|---|---|---|---|---|---|
| ID No. | Location | between | Year Required | Dia (in)[1] | Length (ft)[1] | Cost ($1,000) [2] |
| FS-6 | Fir St | 30th St. and Comanche Dr. | 2005 | 18 | 980 | 270 |
| FS-7 | Fir St | 30th St. and 26th St. | 2005 | 18 | 1,265 | 350 |
| FS-8 | 26th St | Jacqueline Place and Kulshan Avenue | As-Required | 18 | 690 | 190 |
| FS-9 | 26th St | College Way and Kulshan Avenue | As-Required | 12 | 752 | 140 |
| FS-10 | LaVenture Rd | Division St. and Fir St. | As-Required | 10 | 1,525 | 235 |
| FS-11 | LaVenture Rd | Fir St. and Kulshan Ave. | As-Required | 10 | 495 | 75 |
| FS-12 | LaVenture Rd | Fir St. and Kulshan Ave. | As-Required | 12 | 1,386 | 255 |
| FS-13 | Alder Lane Interceptor | Burlington Northern Railroad South of Roosevelt Ave. | As-Required | 24 | 600 | 220 |
| FS-14 | Urban Ave | North of College Way | As-Required | 12 | 375 | 70 |
| FS-15 | Freeway Dr | River Bend Road and Cameron Way | As-Required | 12 | 1,309 | 240 |
| FS-16 | West Mount Vernon | Modify Pump Station | As-Required | | | 150 |
| FS-17 | Central CSO Regulator | Add Fail-safe Gate Operator | 2001 | | | 30 |

1. Improvements are based on saturated development, based on the UGA boundary, 100 gpcd, 1,100 gpad (inflow and infiltration), and L.A. Peaking curve.
2. Costs are based on ENR Cost index of 6390 (October 2001), and include restoration, 25% for legal, administration, and engineering costs, 7.8% for sales tax, and a 20% contingency.




NOTE:

1. IMPROVEMENTS SHOWN HERE ARE THE MINIMUM REQUIRED.

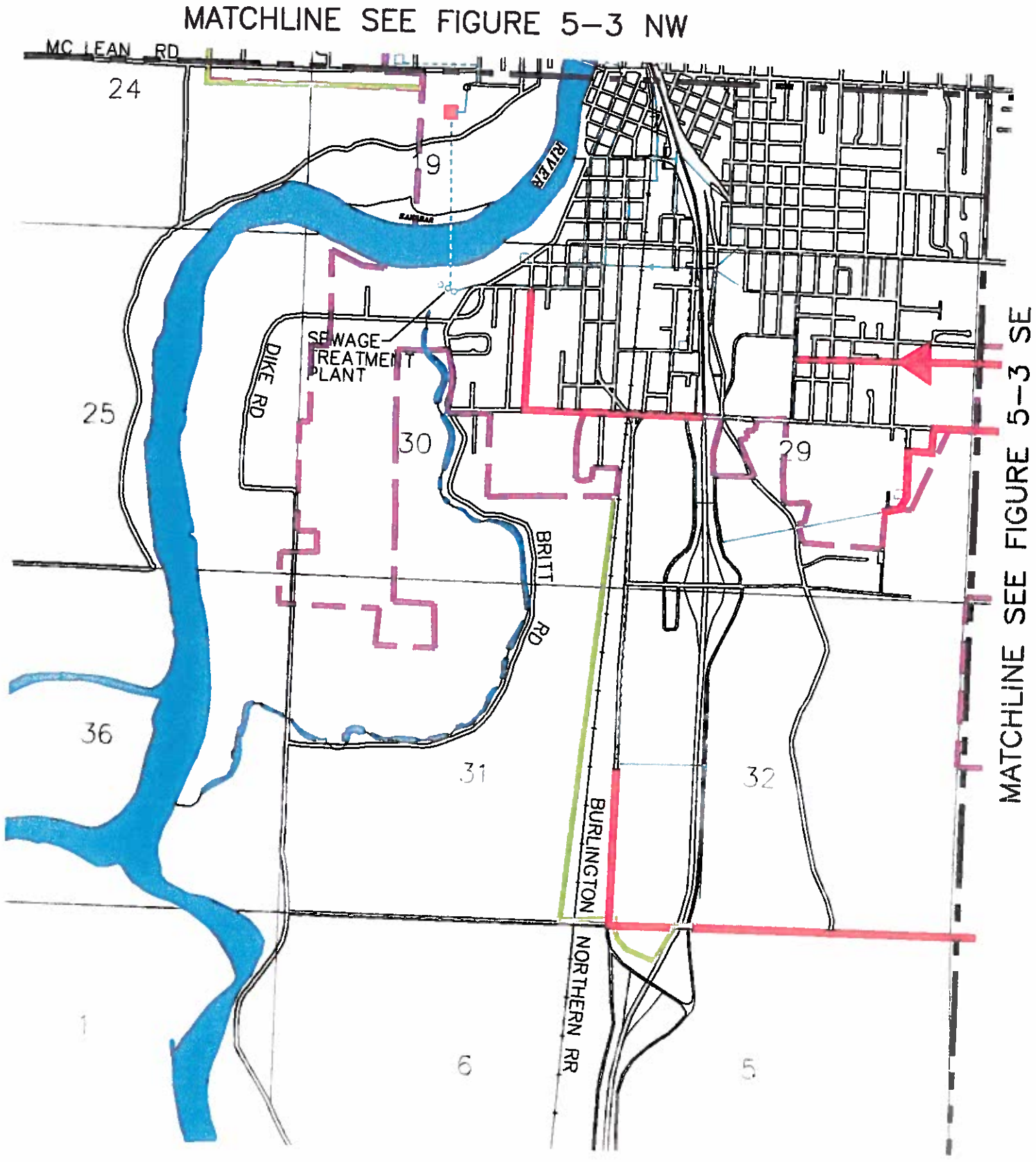
LEGEND:

- FUTURE SYSTEM IMPROVEMENTS
- FUTURE FORCE MAIN
- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROWS)
- EXISTING FORCE MAIN
- EXISTING PUMP STATION
- EXISTING OVERFLOW WEIR
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc.

| | |
|---|---|
| Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE | Date: FEBRUARY 2003 |
| Sheet Title: FUTURE COLLECTION SYSTEM IMPROVEMENTS | Figure No: 5-3 NW |

DATE: 02/13/03




NOTE:

1. IMPROVEMENTS SHOWN HERE ARE THE MINIMUM REQUIRED.

LEGEND:

- FUTURE SYSTEM IMPROVEMENTS
- FUTURE FORCE MAIN
- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROWS)
- EXISTING FORCE MAIN
- EXISTING PUMP STATION
- EXISTING OVERFLOW WEIR
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc.

Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

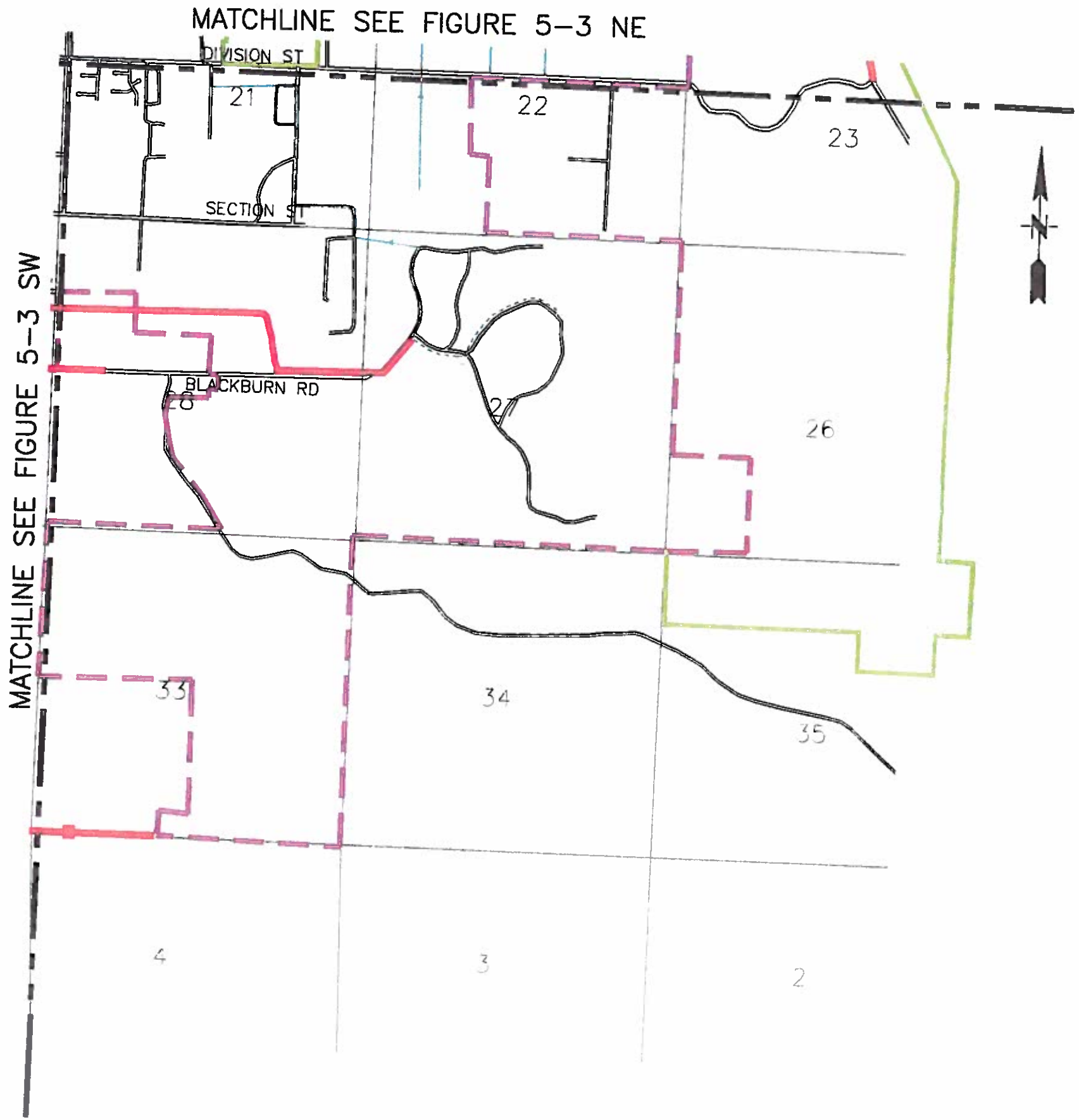
Date: FEBRUARY 2003

Sheet Title: FUTURE COLLECTION SYSTEM IMPROVEMENTS

Figure No.: 5-3 SW

DATE 02/13/03

FILE NAME: FIG5_5SW




NOTE:

1. IMPROVEMENTS SHOWN HERE ARE THE MINIMUM REQUIRED.

LEGEND:

- FUTURE SYSTEM IMPROVEMENTS
- FUTURE FORCE MAIN
- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROWS)
- EXISTING FORCE MAIN
- EXISTING PUMP STATION
- EXISTING OVERFLOW WEIR
- CITY LIMITS
- UGA BOUNDRY

HDR Engineering, Inc.

Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: FUTURE COLLECTION SYSTEM IMPROVEMENTS

Figure No.: 5-3 SF

DATE: 02/13/03

FILENAME: HDR Set

### College Way Pump Station Drainage Area

The 1995 Comprehensive Sewer and Combined Overflow Reduction Plan examined alternatives to conveying flows from the College Way Pump Station to the WWTP via the Kulshan Interceptor. The 1995 recommended alternative, two force mains constructed to the terminus of the Kulshan Interceptor, is still the most efficient method of conveying flows from the existing area and future areas. This alternative recommends flows be conveyed to the Kulshan Interceptor through:

- A new College Way Pump Station, as flows dictate; and
- Two 12-inch force mains from the pump station to the Kulshan Interceptor.

The new College Way Pump Station would convey flows from the UGA (sections 23 and 26), from the eastern portion of the City Limits (sections 15 and 22), and allow the Martin Road Pump Station (see LaVenture Trunk Sewer) to be abandoned. The College Way line from Martin Road to the pump station will need to be upgraded from an 8-inch line to an 18-inch line, with approximately 2,300 LF of pipe. The Martin Road conveyance improvement is accounted for in the improvements to the College Way line, which is undersized for future flows, even without the Martin Road Pump Station flows.

The existing 12-inch line on College Way between Martin Road and 35th Street is predicted near capacity with future development. Current flow data is inconclusive, minor storms recorded may not have fully activated all sources of inflow and infiltration. This line should be monitored every 10 years to determine the affects of growth on flows through this area, but should be monitored more frequently if rapid growth occurs or indications of increases in inflow and infiltration are observed. If necessary, the 12-inch line should be replaced with 548 LF of 15-inch. The existing 8-inch line on Martin Road between College Way and Trumpter Road is predicted near capacity with future development. It should be monitored and replaced with 734 LF of 12-inch line as required.

The estimated peak flow discharged from the College Way Pump Station with a single pump discharge is 960 gpm. The 12-inch line on 26th Street is adequate to accept this single pump discharge, but would be surcharged with 2 pumps operating. Since the line on 26th Street is adequate to accept flows from the College Way Pump Station, alternative to this were not considered.

### Fir Street Trunk Sewer

The Fir Street and 26th Street Trunk Sewers are composed of 8-inch and 12-inch lines. Many of these lines are predicted near capacity with future flows. They should be monitored and replaced as necessary:

- Monitor Fir Street between 30th Street and Comanche Drive and replace with 980 LF of 18-inch pipe, as required.
- Monitor Fir Street between 26th Street and 30th Street and replace with 1,265 LF of 18-inch pipe, as required.
- Monitor 26th Street between Jacqueline Place and Kulshan Avenue and replace with 690 LF of 18-inch pipe, as required.

### LaVenture Trunk Sewer

LaVenture drainage area includes north of Kulshan Creek, along LaVenture, and drainage areas N9 and N15. The existing conveyance includes two pump stations, Hoag Road and Martin Road Pump Stations. As development continues, the interceptor these pump stations discharge to will become overloaded. The Martin Road Pump Station can be abandoned by routing a gravity main from the Martin Road Pump Station to College Way. Martin Road area would be served by a gravity main from the Martin Road Pump Station to the College Way Pump Station, conveying flows via 2,650 LF of new 10-inch pipe and existing lines along College Way from the intersection of College Way and 26th Street to the pump station.

Capacity restrictions in the LaVenture Trunk Sewer exist both north and south of the Kulshan Interceptor. Improvements to the LaVenture Trunk Sewer include both replacement of undersized lines and monitoring of lines predicted to be near capacity:

- Monitor the existing 8-inch line on LaVenture Road between Division Street and Fir Street and replace with a 10-inch line as required.
- Replace the existing 8-inch line on LaVenture Road between Fir Street and Alison Avenue with 495 LF of 10-inch pipe.
- Replace the existing 10-inch line on LaVenture Road between Fir Street and Kulshan Avenue with 1,386 LF of 12-inch pipe.

### Kulshan Interceptor

The Kulshan Interceptor is designed to operate in both a gravity flow and surcharged mode, with a capacity in excess of 20 mgd. Future peak flows will exceed the gravity capacity (9.3 mgd) and the interceptor will operate in a surcharged mode.

### Alder Lane Interceptor

Alder lane Interceptor consists of 30-inch pipes, with a few 15-inch lines. The two sections of 15-inch pipe, paralleling Burlington Northern Railroad, south of Roosevelt Avenue, limit the capacity of the Alder Lane Interceptor. The remaining 30-inch pipe does not result in limitations. These links should be replaced with 600 LF of 24-inch pipe.

The Alder Lane Pump Station currently consists of four pumps with capacities as follows, based on a normal wet well operating level, C factor of 110, and utilizing both the 10 and 16-inch force mains:

- One Pump Capacity: 4.3 mgd
- Two Pump Capacity: 6.8 mgd
- Three Pump Capacity: 8.9 mgd

Peak flows to the pump station in 2020 are estimated at 4.74 mgd. This flow rate will require two pumps, requiring a minimum of three pumps in the station to provide firm pumping capacity.

### Southeast Interceptor

Improvements to the Southeast Interceptor, as identified in the 1995 Comprehensive Sewer and Combined Sewer Overflow Reduction Plan, are different than those recommended in this report, for the UGA boundary has changed in the southern portion of the planning area. Section 34 was included in previous planning studies, but has been omitted from the current UGA. This exclusion changes the predicted future flows and loads entering the Southeast interceptor.

The current mode of operation of the Central CSO Regulator, during periods of high CSO flows, has a beneficial effect of utilizing the Southeast Interceptor for additional storage, yet this could increase the potential that flooding of residences. At projected 2020 flows, of 7.42 mgd, approximately 4.0 ft of headloss to be incurred from the railroad to Hazel Street to the WWTP Influent Pump Station. Depending on the level of downstream surcharging, this level of headloss could cause the hydraulic grade line to be above the ground surface (in affect, sanitary sewer overflows would be possible with downstream surcharging). To prevent this possibility, the following improvements should be implemented prior to increased flows:

- Install a fail safe operator, with a shut mode at failure, at the Harrison Street Vault of the CSO Regulator; and
- Limit the maximum water surface elevation in the influent pump station wet well to 5.5 ft.

### West Interceptor

West Mount Vernon is served by the West Interceptor and West Mount Vernon Pump Station. The analysis predicts no limitations in the West Interceptor, however, it does predict a peak flow of 1.8 mgd in the interceptor. This peak flow is in excess of the firm pumping capacity of the West Mount Vernon Pump Station, 1.2 mgd. Flows from the pump station are conveyed to the WWTP via a 10-inch force main. This force main has adequate capacity for excess of 2.8 mgd.

The West Mount Vernon Pump Station will require upgrade as development approaches saturated conditions on the West side.

This pump station is a 'package-type pump station' with a separate wetwell and drywell. Due to space limitations within the drywell, the most cost effective method of increasing capacity may be to convert this to a submersible pump station, similar to most of the other pump stations within the system. The wetwell would be modified, submersible pumps installed, and a valve vault provided. Budget costs for these improvements and associated electrical improvements are with a standby generator unit is $150,000.

### Central CSO Regulator

The Central CSO Regulator is designed with excess capacity to serve as inline storage during storm events. There are no capacity limitations in this line. A detailed description and analysis of the Central CSO Regulator is presented in Chapter 4.

### Other Trunk Sewer Improvements

Urban Avenue Trunk Sewer, north of College Way, flows are currently conveyed through a 10-inch gravity main. At saturated development, this line is predicted near capacity.

Monitoring of the line is recommended and replacement with 375 LF of 12-inch pipe, as required.

Freeway Drive Trunk Sewer, between River Bend Road and Cameron Way, consists of 8-inch and 10-inch lines. These lines are predicted near capacity with future flows. It is recommended that flow monitoring of these lines occur and replacement with 1,309 LF of 12-inch pipe, as required.

## LOCAL ISSUES

### 1st Street and 8th Street

Many of the sewers in the combined areas are 6 or 8-inch and do not have capacity to convey both sanitary and wet weather flows during extreme storm events. Consequently, backups occur along sections of the sewer that become surcharged during storms. Many of these sewers are over fifty years old and because of deterioration are in need of repair or replacement. One local problem is along North 8th Street between Warren Street and Lawrence. To alleviate the problems in this area the sewers should be replaced with larger sewers as shown in Figure 5-4. The estimated cost for these improvements is $1,000,000.

Where possible the City should consider separating storm water connections from the combined sewer and diverting to storm drainage facilities. Removing the storm water will reduce the peak and volume of flow that is discharged to the treatment plant during storm events. Another option is to provide detention of storm water to reduce the peak discharge rate into the combined system. Separating or detaining flow is particularly beneficial when large areas of impervious surface are removed such as parking lots and large buildings. The City indicated that the Mount Vernon High School is scheduled for renovation. Storm drainage connections from this school could be separated from the combined sewer system or detention structures provided to reduce the peak discharge rate into the combined system.

### Separation of Combined Areas

The 1995 CSO Reduction Plan concluded that it was more cost effective to transport and treat combined sewage rather than separate. The reduction improvements identified in the plan provided a method of conveying the combined sewage to the treatment plant and ultimately treatment of excess flows. This approach to achieving the required level of CSO reduction allows combined areas to remain combined.

The CSO Reduction Plan was developed primarily on the observed peak CSO flow rates for the design storm event and subsequently used to establish the CSO baseline. These flows reflected the extent and nature of development within the combined sewered areas. These areas are almost completely built out and any redevelopment would consist of either reconstruction with the same type of land use such as remodeling a single family residence or possibly a change in the type of land use such as converting single family residential to multifamily residential or commercial. Reconstruction could increase the stormwater runoff

rate and if drainage is provided by the combined sewer system these changes could result in an increase in CSO baseline.

Stormwater design standards, including the City of Mount Vernon's, typically require new construction to maintain predevelopment runoff rates. This requirement protects downstream stormwater facilities from overloading. This same concept and approach could be applied to the combined sewered areas with predevelopment conditions assumed to be those that existed when the CSO baseline for the Reduction Plan was originally established. Requiring redevelopment to provide detention facilities could maintain peak runoff rate into the combined system.

When redevelopment occurs there is the potential for separating storm water connections from the combined sewer and diverting runoff to storm drainage facilities. Even if storm drainage facilities are not available, disconnection of inflow sources such as roof gutter downspouts could benefit the combined system. If downspout splash blocks are provided in areas with no storm drains the runoff would migrate across yards and eventually could enter the combined sewer through right of way inlet connections; however, the rate of flow would probably be attenuated and would reduce the peak flow impact on the sewer. Disconnecting inflow sources such as downspouts also provides the opportunity for the runoff to infiltrate into the ground.

Recent studies indicate that a significant portion of the excess flow in combined sewer systems is from infiltration. Evidence also indicates that much of this flow originates from private property. When redevelopment occurs in combined sewered areas upgrading side sewer laterals to current design standards and excluding subsurface drainage connections such as foundation drains could provide long term benefits of reducing combined sewer flows.

Redirecting runoff in combined sewered areas to storm drainage facilities could also negatively impact the storm sewer system. The existing storm drainage system may not have adequate capacity to accommodate the additional runoff. Furthermore, increasing the runoff to a storm drainage system from previously combined sewered areas may hamper efforts to maintain water quality of stormwater runoff.

The City should further evaluate the impacts of increased runoff into the combined system from redevelopment and the impacts of separating sewers in the combined areas.

### Interstate 5 Crossing

There are several sewer crossings under Interstate 5 that are damaged and need to be replaced or repaired. The repair method will be challenging for the crossings between Kincaid Street and the 2nd Street Overpass because the lines are behind a large retaining wall on the east side of the freeway. The sewers that should be addressed are described below; however, each crossing should be evaluated further to determine the most appropriate repair method. Repair or replacement methods include bore and jack, cure in place, pipe burst, horizontal directional drill, or other rehabilitation technologies may be possible. The estimated costs for repairing all of the Interstate 5 crossings is approximately $750,000, assuming cure in-place lining of existing pipes. See Table 5-3 and Figure 5-5 for the location of the sewers.

**Table 5-3**

| Interstate 5 Crossings | | |
|---|---|---|
| No. | Location | Condition and Recommended Improvement |
| 1. | Lawrence Street to old Brick Hill Overflow Structure | Condition is unknown. The line should be evaluated and repaired, replaced or lined as necessary. The sewer maps indicate that there is a manhole located on this freeway crossing in the middle of Interstate 5. If the improvements identified in the North 8th Street discussion are constructed the flows in this freeway crossing will be reduced. |
| 2. | Fulton Street to Freeway Drive near Scotts Bookstore | Condition is unknown. The line should be evaluated and repaired, replaced or lined as necessary. The pipe serves an extremely small area so lining the pipe may be desirable. The sewer maps indicate that there is a manhole located on this freeway crossing in the middle of Interstate 5. |
| 3. | From 4th Street dropping under the 2nd Street Overpass | Video tapes of the pipe indicate that the pipe is damaged. The 2nd Street overpass is scheduled to be replaced. This sewer crossing could be suspended from a new bridge. Houses immediately adjacent to the bridge should be evaluated to determine if they can be served by a new suspended bridge crossing. |
| 4. | Division Street | Condition is unknown. The line should be evaluated and repaired, replaced or lined as necessary. It is possible the flow in this line could be routed north to the 2nd Street Overpass crossing. |
| 5. | 4th Street and Washington | Video tapes of this sewer pipe have documented damage. The line should be reevaluated and repaired, replaced or lined as necessary. It is also possible that a sewer line could be constructed in the east shoulder of the freeway to intercept these flows and route them south to Kincaid Street. |
| 6. | From Gates Street on the West Side of the Freeway to the Kincaid Street Northbound onramp | This crossing was abandoned during the construction of the Central CSO Regulator. |
| 7. | 6th Street and Gates on East Side of Freeway | Condition is unknown. This line crosses under the Kincaid Street Northbound onramp and then flows south to Kincaid. The line should be evaluated and repaired, replaced or lined as necessary. |

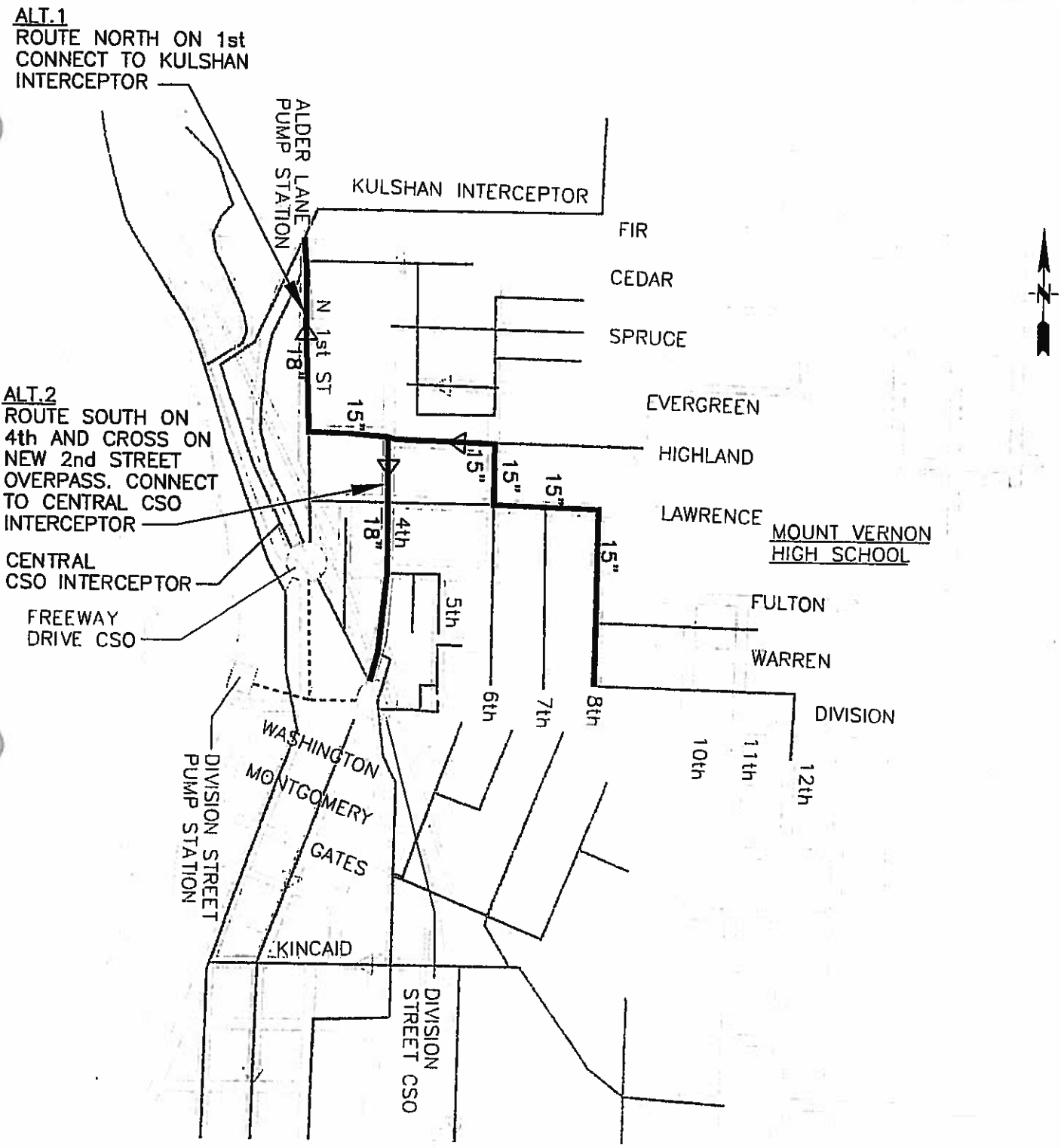
| Interstate 5 Crossings | | |
|---|---|---|
| No. | Location | Condition and Recommended Improvement |
| | | necessary. |
| 8. | Section Street at Wells Nursery | Condition unknown. This 16-inch provides service to only one connection, Wells Nursery. There is also a documented steady flow |
| 9. | Park Street at South Side of Wells Nursery | Condition unknown. The line should be evaluated and repaired, replaced or lined as necessary. |

### North Fir Street

As development occurs in the property East of 30th Street and North of Division Street conveyance will be required. Conveyance from this area should be connected to the line on 30th Street. The line should be extended up to Division to intercept and offload other local sewers. This extension could also provide service to a future school East of 34th Street and South of Division Street.

### Fowler Interceptor

Wastewater from the Eaglemont Development in East Mount Vernon currently discharges to the north and flows to the Kulshan Interceptor. Original plans for this development identified the need to ultimately convey flows to the Fowler Interceptor. This interceptor has been extended partially to the east already. The remainder of the extension should be completed as required by the development of Eaglemont.




HDR Engineering, Inc.

Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

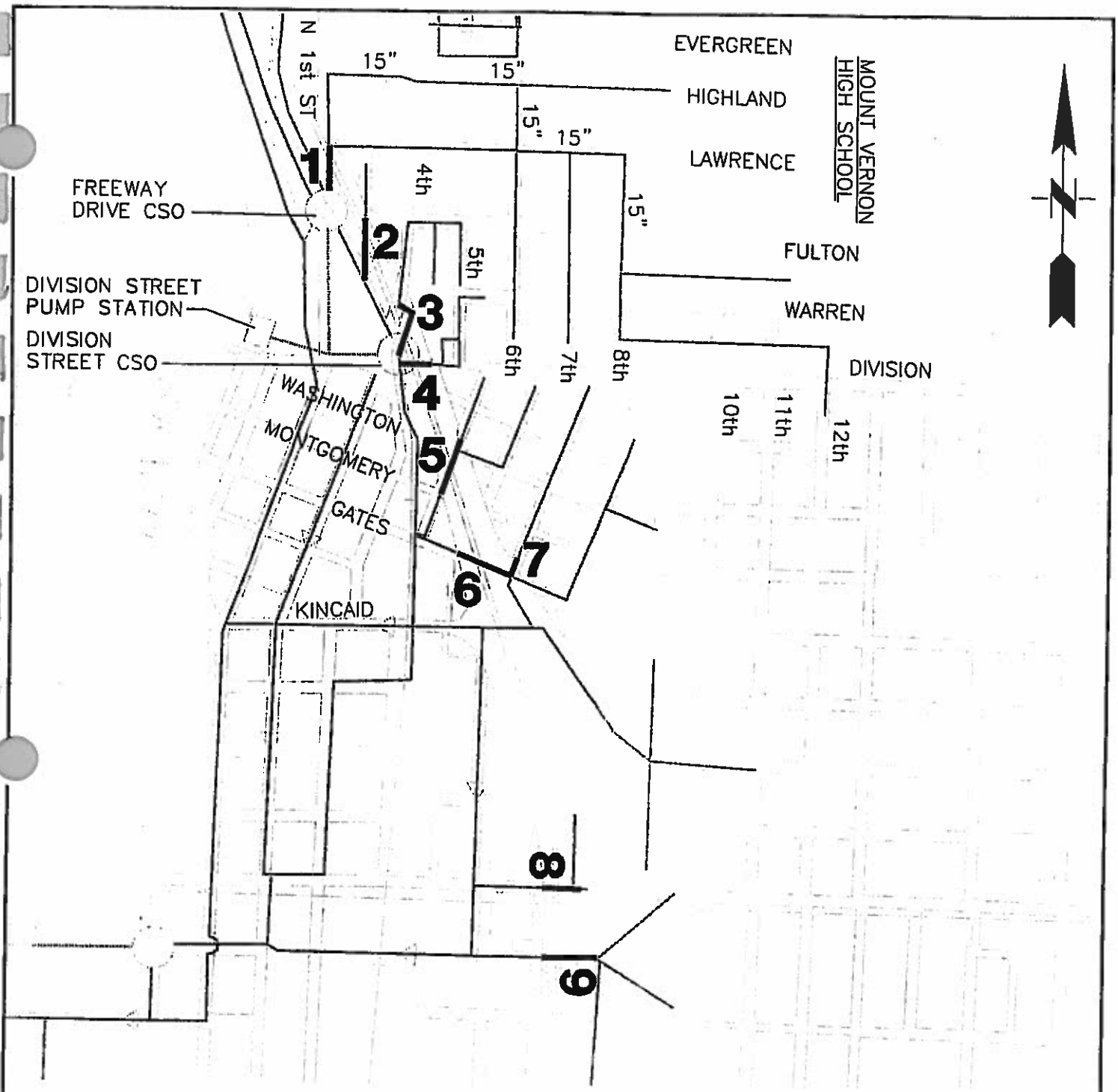
Date: FEBRUARY 2003

Sheet Title: NORTH 8th STREET IMPROVEMENTS

Figure No. 5-4

DATE: 02/12/02

FILENAME: FIG5-7




HDR Engineering, Inc.

| | |
|---|---|
| Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE | Date: FEBRUARY 2003 |
| Sheet Title: INTERSTATE 5 SEWER CROSSINGS | Figure No. 5–5 |

### Freeway Drive Pump Station

This pump station serves the limited development on the west side of Interstate 5 between College Way and the Skagit River. The pump station has adequate capacity to serve the boundaries and current zoning. Any revisions to the zoning or expansions of the service area may require an upgrade to the pump station. The existing pump station and 8-inch force main have a capacity of about 350 gpm. This is about 2 feet per second velocity in the force main. It is reasonable to increase velocities in a force main to about 8 feet per second so additional capacity could be provided by increasing the pumping rate. The sewer beyond the force main discharge may need to be increased to accommodate additional flows.

### South Mount Vernon

Service to the area of Anderson Road has been provided by constructing a pump station on Highway 99 South of Anderson Road. Areas on the East side of Interstate 5 will be served by a gravity sewer extending under Interstate 5 approximately halfway between Anderson Road and Hickox Road. There is a small area of south of Little Mountain Park that will need to be provided with a pump station because the grade falls to the east.

## WASTEWATER COLLECTION SYSTEM DEFECTS ASSESSMENT

### Introduction

The City has three databases that are used to track sewer collection system problems:

- Video Scan, a database record of the TVing of sewer lines;
- Sewage Incident Reports, a database of incidents of water and wastewater on the ground; and
- Sewer Complaints, a database of customer complaints of suspected waters that may or may not be wastewater, and of local problems (i.e. wastewater flooding basement due to plugged side sewer).

Table 5-4 lists major defects identified through the City video records and system database. The City has also compiled a database of customer reported problems, sewage incidents, and historical video inspections. System deficiencies included deteriorating pipes, lines with excessive root intrusion, or lines known to have capacity limitations. Minor defects that can be addressed with spot fixes are discussed in the next section.

Table 5-4

| Collection System Improvements | | | | | |
|---|---|---|---|---|---|
| ID No. | Location | Defect | Defect identified Via | Improvement | Cost ($1,000)[1] |
| CS-1 | Snoqualmie, MH B29A to MH B29 | Root intrusion | Video[2] | Remove roots and Slipline with 300 LB | $20 |
| CS-2 | Yard of house 1115 N0. 8th, MH 49 to MH 50 | Root intrusion | Video[2] | Remove roots and Slipline with 250 LB | $20 |
| CS-3 | So. 7th and Jefferson to So. 7th and Washington, MH 39 to MH 37 | Root intrusion | Video[2] | Remove roots and Slipline with 450 LB | $20 |
| CS-4 | No. 6th and Lawrence, MH C39 to MH C38 | Root intrusion | Video[2] | Remove roots and Slipline with 320 LB | $20 |
| CS-5 | Brick Hill, MH 01, North along I-5 | Root intrusion | Video[2] | Remove roots and Slipline with 400 LB | $20 |
| CS-6 | Blodgett Rd to North of Blackbur, MH 55 to MH 54 | Root intrusion | Video[2] | Remove roots and Slipline with 270 LB | $20 |
| CS-7 | Kincaid, MH 25, to MH 23 | Root intrusion | Video[2] | Remove roots and Slipline with 240 LB | $20 |
| CS-8 | So. 20th, North off Section, MH 32 to MH 31 | Root intrusion | Video[2] | Remove roots and Slipline with 120 LB | $20 |
| CS-9 | Section, MH D33 to between MH D32-D31 | Structural Damage | Video[2] | Replace with 420 LF of 8-inch pipe | $50 |
| CS-10 | Alley between Douglas and Walter, MH A13 to A05 | Structural Damage | Video[2] | Replace with 640 LF of 8-inch pipe | $75 |

| Table 5-4 cont | | | | | |
|---|---|---|---|---|---|
| ID No. | Location | Defect | Defect identified Via | Improvement | Cost ($1,000)[1] |
| CS-11 | 107 Cedar to the South, MH F11 to F29 | Structural Damage | Video[2] | Replace with 300 LF of 8-inch pipe | $45 |
| CS-12 | No. 6th, MHF13 to F14 | Structural Damage | Video[2] | Replace with 400 LF of 8 | $60 |
| CS-13 | Section and Rail Road Ave, MH E17 to E18 | Structural Damage | Video[2] | Sport repair-verify grease problem is corrected | $5 |
| CS-14 | Broadway at alley between So. 9th & 10th, MH D41 to D40 | Structural Damage | Video[2] | Slipline with 330 LF | $20 |
| CS-15 | Broad, east of So. 11th, MH 54 to MH 49 | Structural Damage | Video[2] | Replace with 230 LF of 8-Inch pipe | $20 |
| CS-16 | Line under I-5 | Structural Damage | Video[2] | Will require further | --[4] |
| CS-17 | Alley, north of DIvision, east of No. 11th, MH C66 to C65 | Structural Damage | Video[2] | Spot Repair | $5 |
| CS-18 | Bernice, east of So. 14th, MH G42 to G41 | Structural Damage | Video[2] | Spot Repair | $5 |
| CS-19 | So. 3rd and Vera, MH A41 to I42 | Structural Damage | Video[2] | Pipe has been | -- |
| CS-20 | Lawrence and 7th, MH C73 | Structural Damage | Video[2] | Spot Repair | $5 |
| CS-21 | 1224 12th Str. So, between MH G8 and G11 | Structural Damage | Video[2] | Replace with 200 LF of 8-inch pipe | $25 |

| | | | | | |
|---|---|---|---|---|---|
| | and G11 | | | inch pipe | |

**Table 5-4 cont**

| ID No. | Location | Defect | Defect identified Via | Improvement | Cost ($1,000)[1] |
|---|---|---|---|---|---|
| CS-22 | 117th North 8th Str. | Flooding | Data Base[3] | See 8th Str. Section[3] | --[5] |
| CS-23 | 420 E. Fulton | Flooding | Data Base[3] | See 8th Str. Section[3] | --[5] |
| CS-24 | 919 W. Division | Flooding | Data Base[3] | No improvements-surface flooding problem | -- |
| CS-25 | Alley at Carpenter, between So 9th and so. 10th heading north to Division | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-26 | 1120 No 16th, 340 ft north of MH M68 on Florence and 16th | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-27 | 1210 N. 14th, north of Florence and 14th | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-28 | 8th Str. And Evergreen heading north, F18 to F15 | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-29 | 7th and Warren, toward Fulton, MH C73 to C72 | Cracked Pipe | Data Base[3] | See 8th Str. Section | --[5] |
| CS-30 | 16th and Blackburn heading east 17th, J08 to J09 | Obstruction | Data Base[3] | Jet main and monitor flows | -- |
| CS-31 | 100 Washington-storm line going to SE under I-5, MH C19 to C20 | Cracked Pipe | Data Base[3] | Will require further assessment | --[4] |

| Table 5-4 cont. | | | | | |
|---|---|---|---|---|---|
| **ID No.** | **Location** | **Defect** | **Defect identified Via** | **Improvement** | **Cost ($1,000)[1]** |
| CS-32 | Scott's Bookstore, N 1st to N 1st and Division | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-33 | Snoqualmie St. between Cleveland and S 2nd Str. MH B32 to B03 | Cracked Pipe | Data Base[3] | Reassess slipline if necessary | -- |
| CS-34 | Westside of Christenson Seed West to So 3rd, MH E01 to A39 | Infiltration | Data Base[3] | Spot Repair | $5 |
| CS-35 | Cleveland and Blackburn to just West of Harrison and Blackburn, MH J11 to J09 | Inflitration, Joint problem | Data Base[3] | Slipline 300 LF | $20 |
| CS-36 | N Laventure just south of E Fir to N Laventure just north of E Fir, MH N06 to N04 | Root intrusion | Data Base[3] | Reassess slipline if necessary | -- |
| CS-37 | North of Casecade Str., on N Laventure to S of E Fir on Laventure, MH N08 to N06 | Root intrusion | Data Base[3] | Reassess slipline if necessary | -- |
| CS-38 | N Laventure, Fulton to Cascade, MH N12 to N10 | Cracked Pipe | Data Base[3] | Spot Repair | $5 |
| CS-39 | Hoag Rd., Parkway Dr., to Hoag Rd | Root intrusion | Data Base[3] | Reassess slipline if necessary | -- |
| CS-40 | Lind Str. And S. 6th to N on S 6th, MH E76 to E75 | Infiltration | Data Base[3] | Spot Repair | $5 |

[1] Costs are based on ENR Cost Index of 6390 (October 2001), and include restoration, 25% for legal, administration, and engineering costs, 7.8% for sales tax, and a 20% contingency.
[2] Defect identified via review of video records.
[3] Defect identified via review of City Sewer Data Base.
[4] Interstate-5 Crossings are estimated at $750,000 for all nine improvements.
[5] 8th Street improvements have been estimated at $1,000,000 to correct the localized surcharging.

### Repair and Replacement Program Criteria

The City has annually allocated a budget of $900,000 for sewer repair and replacement. This allocation allows necessary improvements to be scheduled and completed in a timely manner, saving the City monies from costly emergency repairs. Co-ordination with the Pavement Management Plan also allows savings to be realized for the City. A comprehensive repair and replacement program, designed to address improvements in order of importance, is presented in the last section of this chapter, Recommendations.

The City databases were reviewed and the necessary capital improvements identified. Numerous problems are small in nature and can be repaired with spot fixes. These defects should be allotted a nominal sum of $5,000 per location for repair of the problem. Defects that require additional work, including removing roots and sliplining have been allocated a minimum estimated cost of $20,000. The City databases had in excess of 30 records where repairs were required. Table 5-4 presents a summary of the identified projects and the corrections for each problem.

## ODOR CONTROL

Odors in the collection system are typically associated with anaerobic conditions. These conditions are a function of ambient temperature, gravity pipe slope, transition structures, inverted siphons, and force mains. Hydrogen sulfide is generated in the wastewater and released to the atmosphere, causing odors and corrosion in the structure where it is released. Typically, in the collection system, prevention or treatment of hydrogen sulfide in the liquid-stream is desirable.

Liquid-stream odor control can be accomplished by numerous chemicals:

- Chlorine, as is currently utilized, is a powerful oxidant that can be supplied either in a gas phase (chlorine gas) or as hypochlorite. It is effective at controlling odors by oxidizing sulfide and killing or inactivating many odor-causing bacteria. Chlorine oxidation requires approximately ten to fifteen pounds of chlorine per pound of sulfide. It's key disadvantage is it's classification as a hazardous substance, which requires consideration of health and safety issue.

- Calcium nitrate is an alternate electron donor. In anaerobic conditions, bacteria preferentially chose nitrate to sulfate as an electron donor, thus sulfide is not produced in the presence of nitrate. Approximately 0.7 to 1.4 pounds of calcium nitrate is required per pound of hydrogen sulfide. Bioxide™ is a commercially available calcium nitrate solution produced by U.S. Filter, Davis Process.

- Other options for chemical oxidation of sulfide include potassium permanganate, hydrogen peroxide, ferrous sulfate, and slug dosing with caustic.

Four options were reviewed for reducing odors in the collection system. These included oxidizing with potassium permanganate, sodium hypochlorite, gaseous chlorine, and the addition of calcium nitrate. Typical costs per pound of sulfide removed were developed for each of these options.

| Item | Cost per lb of Sulfide Removed |
|---|---|
| Potassium Permanganate | $7 - $10 |
| Sodium Hypochlorite (12%) | $3 - $5 |
| Gaseous Chlorine | $1 - $3 |
| Calcium Nitrate | $2 - $3 |

Although gaseous chlorine has the lowest cost per pound of sulfide removed, the handling of gaseous chlorine presents a number of safety related issues, as addressed in Article 80 of the Uniform Fire Code. This requires the provision of containment and scrubber system to treat gases that could leak from the system. Due to the additional regulations and safety concerns, the trend for many utillties is to avoid the use of gaseous chlorine when planning new facilities. Presently the City utillzes the gaseous chlorine system at the wastewater treatment plant to provide a chlorine solution that is pumped to the incoming interceptor of the wastewater treatment plant at Hazel Street and Harrison Street. If gaseous chlorine were not to be used in the future, the use of calcium nitrate would be the next most cost effective method for odor control.

The future plan would be to add calcium nitrate at the more remote locations in the collection system, thereby reducing the production of hydrogen sulfide within the system and the need to add large quantities of chlorine to the interceptor upstream of the wastewater treatment plant.

## RECOMMENDATIONS

While some deficiencies in the collection system exist or will exist with projected future growth, not all of them are recommended for repair or replacement. Table 5-5 presents the recommended improvements and a schedule for implementation, correlating to priority of improvement. Improvements to the interceptor system are dependant upon future growth and should be constructed, as identified in Table 5-2, to serve the areas that experience growth.

## Table 5-5

| Repair and Replacement Program | | |
|---|---|---|
| **Year(s)** | **ID Tags** | **Cost ($1,000)** |
| 2001 | CS-1 through CS-18, CS-20, CS-21, CS-25 through CS-28, CS-32, CS-34, CS-35, and CS-40[1] | $555 |
| 2002 | FS-5[2] | $635 |
| 2003 | 8th Street Improvements | $1,000 |
| 2004 | Interstate 5 Crossings | $750[3] |
| 2005 | FS-6 and FS-7[2] | $620 |
| 2006 | Interceptor Improvements | -[4] |
| 2007-2020 | FS-1 through FS-4, and FS-8 through FS-17[2] | $2,540 |
| Total | | $6,100 |

1. Improvements identified by the City, Table 5-4.
2. Interceptor System improvements identified in Table 5-2.
3. Interstate 5 Crossings improvements are identified in Table 5-3.
4. The interceptor improvements identified in Table 5-2, and accounted for in this table in the future (2007-2020) should be designed and constructed as growth dictates.

# 6. INDUSTRIAL PRETREATMENT

## INTRODUCTION

The City of Mount Vernon has one major industrial customer, Draper Valley Farms, Inc. (DVF), which discharges to the City's wastewater collection system. This industrial discharge is regulated by a State Discharge Permit, issued by the State of Washington Department of Ecology (DOE). This permit defines pretreatment requirements for these wastewater discharges to the City's sewer system.

As a part of the comprehensive planning process, the operations at this industry and their pretreatment equipment were reviewed to determine the adequacy of the pretreatment being provided. This included onsite observation of the industrial operation, interviews with operating staff, a review of operating data and compliance with permit requirements, and recommendations for operational plant modifications or improvements to the pretreatment process. This chapter includes description of the poultry plant and associated pretreatment facilities, presentation of wastewater data and wastewater discharge limitations, and a discussion and conclusions regarding the DOE requirements for the processes meeting the criteria for 'All Known, Available, and Reasonable Methods of Treatment (AKART).'

## POULTRY PLANT DESCRIPTION

Draper Valley Farms slaughters approximately 90,000 fryer/broiler chickens during two productions shifts. The plant normally operates five days per week with some six-day weeks and one seven-day week each year, at most. The plant is sanitized during the third shift, with an additional "pre-operation" cleanup that starts at midnight on Sundays.

Cooling fans are activated in the receiving area when temperatures reach 65 °F; while misters are activated when temperatures reach 70 °F. After the chicken cages are unloaded, pretreated wastewater is recycled to wash the cages before they are returned to the truck.

After the carotid artery of the chicken is cut, the blood is collected in a curbed area and pumped to a holding tank on one of the trucks that hauls inedible material to the off-site renderer. The birds are scalded with steam to allow removal of yellow skin in the plucking machines to yield regionally-desirable white broilers, rather than yellow broilers. Feathers, and the yellow skin, are removed in three mechanical plucking machines in series, with the final machine devoted to feet of the bird. The feathers and skin are directed to one of two inedible trucks. Later the feet are removed and, somewhat unusually, sold as edible product in the United States. Guts, lungs, crops, heads and other inedible materials are directed to a second inedible truck. Giblets are removed and chilled with water for sale. Ultimately the chickens enter a chiller where heat is removed from the carcass with cold water. After chilling, some of the carcasses are directed to an adjacent room for cutting and packaging.

The entire production area is equipped with good areas designated for washing aprons and hands. The use of these areas during breaks, noon and shift changes prevents washing material on the floor into the sewers before it can be removed by dry cleaning.

All refrigerant compressors are air cooled, while cooling towers are used for the ammonia and freon compressors. Water is periodically blown down from the cooling towers to the plant with an automatic timer to prevent a buildup of minerals. This blow down is directed to the plant sewers through a one-inch line.

## PRETREATMENT FACILITIES

Wastewater pretreatment facilities consist of primary and secondary screening and dissolved air flotation (DAF) with chemical addition. After feathers are plucked from the birds they drop Into a flume for conveyance to the feather screen. This screen is a rotating, internally-fed screen with openings approximately 1/8 inch in size. Feathers are sent to a press for dewatering and then augured to a truck for hauling to the off-site renderer. Viscera, heads, and other offal drop into a flume for conveyance to the offal screen. This screen is also a rotating, internally-fed screen with openings approximately 1/8 inch in size. Screened offal is augured to a compartment in the inedible truck, separate from the feathers. Underflow from the feather and offal screens is recycled with a pump back to the head end of the feather flume for conveying the feathers. This recycling is acceptable in the feather plucking area, but would not be acceptable in the remainder of the plant after the bird carcasses have been opened. Therefore USDA-required overflow water from the chiller, and other flows from the various processing operations, is utilized to convey the inedible material in the offal flume to the offal screen.

Screen underflow enters a wet pit. In addition to the recycle pump for the feather flume, this wet pit is equipped with a mechanical mixer and three submersible pumps. These three pumps are used to pump the wastewater through three individual forcemains to a secondary screen, although two of these pumps can handle the entire flow, even during the peak hydraulic flow period when the chiller tank is dumped. The secondary screen is a rotating internally-fed screen, with 0.02-inch openings. Screenings from this screen are combined in the inedible truck with the offal.

Since November 1999, a combination of ferric chloride and acid has been injected into each of the three lines to the secondary screen. A pH controller ensures a sufficient quantity of this liquid is added to reduce the pH to approximately 4.1 to 4.5. This pH range is the approximate isoelectric (point of least solubility) point of the proteins in the wastewater. After the excess proteins have come out of solution, they are coagulated by the ferric (trivalent iron). Polymer is then added to flocculate the coagulated proteins before the secondary screen underflow enters the subsequent DAF tank.

The above-ground steel DAF tank is approximately 70 ft long, 10 ft wide and 8 ft high, including 6 inches of freeboard. As such, it holds approximately 39,400 gallons. At the maximum allowable daily flow of 630,000 gpd, this results in a detention time of nearly 90 minutes. Secondary screen underflow is divided between four equally-spaced, 8-inch influent lines near the head end of this tank. To create a dissolved air flotation system, a portion of the tank contents is pumped from a line about a foot off the bottom and midway down the tank. A controlled amount of atmospheric air is aspirated into the suction line to

this 15-hp recycle pressurization pump. The pump discharge is divided into four lines, each equipped with a back-pressure valve before it combines with one of the DAF influent lines. To drive most of aspirated air into solution, the valves are throttled to yield a back-pressure approximately 90 psi. After passing through the back-pressure valve and combining with the flocculated screen underflow, the dissolved air comes out of solution as small bubbles which attach to flocculated solids to float them to the surface of the DAF tank. Somewhat unusually, four large fans are periodically activated to blow the floating solids to the effluent end of the tank where they are swept into a skimmings hopper with a large paddlewheel. Occasionally, however, the operator has to assist the fans by raking the floating solids to the paddlewheel. After a quiescent period, water is drained from these skimmings and then they are pumped, with an air-operated, double-diaphragm pump to a separate compartment on one of the inedible trucks. After this skimmings compartment becomes full, the remaining skimmings are pumped to a separate skimmings tanker truck. The DAF tank is not equipped with any positive means of settled solids removal; however, the location of the recycle pump suction near the bottom of the DAF tank tends to draw some of these solids off the tank bottom. Nevertheless, a settled sludge layer varying from six inches to two feet had accumulated on the tank bottom when this tank was recently drained for the first time after more than five years.

A reuse pump is located near the DAF recycle pressurization pump to supply DAF tank contents for the initial hose down of the chicken cages and for hosing down the pretreatment and inedible truck areas.

DAF effluent overflows a relatively-short weir plate into a collection launder at the effluent end of the DAF tank. A pH sensor is used to regulate the feed of sodium hydroxide solution to maintain the pH of the effluent in the range of 6 to 7. Pretreated effluent is directed through a sampling and metering manhole before it enters the City sewer system. A 10-inch Palmer Bowlus flume with an ultrasonic level sensor is used to pace a ISCO refrigerated composite sampler. Wastewater billings are based on potable water meter readings, however, because the flume would surcharge in the past when flows exceeded 0.6 mgd.

## WASTEWATER DISCHARGE LIMITATIONS

The Washington Department of Ecology (WDOE) has issued a discharge permit for Draper Valley Farms to discharge pretreated wastewater to the City of Mount Vernon sewerage system. This permit is effective until May 29, 2003. Effluent limits contained in this permit are:

| EPA Recommendations | Draper Valley Farms Practices |
|---|---|
| | 2. Water to wash station before eviscerating and after cavity splitter ran during noon break. |
| | 3. Water to eviscerating machine ran during noon break. |
| | 4. Water sprays at conveyor into carcass chiller ran during noon break. |
| | 5. Shower at the salvage station wired open to flow continuously. |
| • Use minimum USDA-approved quantities of water in the scalder and chillers. | • Yes. Checked twice each shift |
| • Control water use in gizzard splitting and washing equipment. | • Yes |
| • To reduce the waste loads, dry clean all floors and tables prior to washdown in: | |
| • Bleeding. | • Yes |
| • Cutting. | • Yes |
| • Further processing areas. | • Not applicable |
| • All other areas where there tend to be material spills. | • Yes |
| • Minimize the amount of chemicals and detergents to prevent emulsification or solubilizing of solids in the wastewaters. | • Yes |
| • Stun birds in the killing operation to reduce carcass movement during bleeding. | • Yes |
| • Confine bleeding and provide for sufficient bleed time. | • Yes |
| • Recover all collectable blood and transport to rendering in tanks rather than by dumping on top of feathers or offal. | • Yes |

| EPA Recommendations | Draper Valley Farms Practices |
|---|---|
| • Consider the reuse of chiller water as makeup water for the scalder. | • No. Rarely, if ever, done in large, modern poultry plants |
| • Consider steam scalding as an alternative to immersion scalding. | • Not acceptable to USDA that requires 1 quart of water per bird be used in the scalder. |
| • Recycle screened wastewaters for feather fluming. | • Yes |
| • Consider dry offal handling as an alternative to fluming. | • No. Rarely, if ever, done in large, modern poultry plants |
| • Control inventories of raw materials used in further processing so that none of these materials are wasted to the sewer. Spent raw materials should be routed to rendering. | • Not applicable – no further processing |
| • Treat separately all overflow of cooking broth for grease and solids recovery. | • Not applicable – no cooking at this plant |
| • Reduce the wastewater from thawing operations. | • Not applicable – no thawing at this plant |
| • Treat offal truck drainage before sewering. One method is to steam sparge the collected drainage and then screen. | • No. Rarely, if ever, done in large, modern poultry plants |
| • Avoid overfilling cookers in rendering operation. | • Not applicable – no rendering operation |
| • Provide and maintain traps in the cooking vapor lines of rendering operations to prevent overflow to the condensers. This is particularly important when the cookers are used to hydrolyze feathers. | • Not applicable – no rendering operation |
| • Use pretreated poultry processing wastewaters for condensing all cooking vapors in onsite rendering operations. | • Not applicable – no rendering operation. |
| • Provide bypass controls in rendering operations for controlling pressure reduction rates of cookers after feather hydrolysis. | • Not applicable – no rendering operation |

| EPA Recommendations | Draper Valley Farms Practices |
|---|---|
| • Stop cooker agitation during cooker pressure bleed-down to prevent or minimize materials carry-over. | • Not applicable – no rendering operation |
| • Provide frequent and regularly scheduled maintenance attention for byproduct screening and handling systems throughout the operating day. | • Yes |
| • Provide a back-up screen to prevent byproduct from entering municipal waste treatment system. | • No. Rarely, if ever, done in large, modern poultry plants |
| • In-plant primary systems—catch basins, skimming tanks, air flotation, etc. - should provide for at least a 30-minute detention time of the wastewater. | • Yes – closer to 90 minutes |
| • Provide frequent, regular maintenance attention to air flotation system. | • Yes |
| • Dissolved air flotation with pH control and chemical flocculation. | • Yes |

Methods of "prevention, control and treatment" of wastes discharged from a poultry plant to a municipal treatment system include the following general categories:

- In-plant waste minimization
- Recycle/reuse
- Pretreatment

The previous comparison shows that DVF has implemented virtually all the applicable BPT, New Source and BAT technologies suggested by the EPA for in-plant waste minimization, recycle/reuse and pretreatment, at least as currently practiced by large, modern poultry plants. DVF's recycle and reuse practices are unusually good.

AKART pretreatment requirements cannot be defined for a poultry plant without taking into consideration the municipal wastewater treatment facilities, since wastes can be removed at either location. Some municipalities have expanded their wastewater treatment facilities to accommodate waste loads from poultry plants with physical pretreatment alone, while many cities have required poultry plants to meet discharge limits around domestic strength levels, often around 250-350 mg/L $BOD_5$ and suspended solids (TSS). These domestic strength limits are about a quarter to a third of discharge levels with physical pretreatment alone. The current $BOD_5$ concentrations discharged by DVF to the sewer system are 200 to 250

mg/L on a 3 day average. The following is a listing of wastewater pretreatment options for poultry plants, arranged from least effective to most effective:

1. Coarse (1/4" openings) screening.
2. Coarse and fine (0.02" to 0.04" openings) screening.
3. Coarse and fine screening and gravity clarification.
4. Coarse and fine screening and dissolved air flotation.
5. Coarse and fine screening and dissolved air flotation with cationic polymer addition.
6. Coarse and fine screening and dissolved air flotation with cationic and anionic polymer addition.
7. Coarse and fine screening and dissolved air flotation with alum and anionic polymer addition with subsequent caustic addition for effluent pH neutralization, if required.
8. Coarse and fine screening, dissolved air flotation with acidulation to the isoelectric point (pH of least solubility of proteins) and polymer addition for protein coagulation and flocculation with subsequent caustic addition for effluent pH neutralization.
9. Coarse and fine screening and dissolved air flotation with ferric and anionic polymer addition with subsequent caustic addition for effluent pH neutralization, if required.
10. Coarse and fine screening, 24-hr flow equalization, dissolved air flotation with ferric and anionic polymer addition, effluent turbidimeter with provisions to return off-spec effluent back to the 24-hr flow equalization basin (FEB) and caustic addition for effluent pH neutralization, if required.
11. Coarse and fine screening, 24-hr flow equalization, dissolved air flotation with ferric and anionic polymer addition, effluent turbidimeter with provisions to return off-spec effluent back to the 24-hr FEB, caustic addition for effluent pH neutralization, and a 7-day FEB.

After the maximum amount of physical pretreatment, consisting of coarse and fine screening and dissolved air flotation, is achieved, further poultry waste reductions are almost always accomplished with chemical addition. The least effective chemicals for pretreatment yield the most acceptable sludges for rendering. Conversely the most effective chemical for pretreatment, ferric sulfate/chloride, yields a sludge which is difficult to render and seriously degrades the rendered products. Nevertheless DVF uses ferric chloride to meet the required discharge limits. In fact, they also acidulate the wastewater to the isoelectric point for even greater removals. Flow equalization ahead of the chemical pretreatment, monitoring effluent quality and return of off-spec wastewater for retreating, and 7-day flow equalization are additional steps that can be taken to improve the consistency of pretreatment, if necessary. The data shown in Table 6-1 shows the effluent has consistently met the discharge limits after the initial start-up of the new chemical feed system.

## POTENTIAL IMPROVEMENTS

Although DVF is meeting the requirements of AKART in discharging their pretreated wastewater to the City of Mount Vernon's wastewater treatment system, there are a few enhancements that DVF should consider:

### In-Plant Waste Minimization

1. Replace home shower-type nozzles with engineered spray nozzles.
2. Evaluate automating the flow of potable water to the plucking machines, eviscerating machine, and conveyor to the carcass conveyor so it shuts off automatically at noon and during breaks when there are no birds passing through these devices.
3. Continue to train, encourage and monitor plant personnel to turn off water at work stations during breaks and at noon.
4. Continue to ensure all hoses are equipped with press-to-activate nozzles.

### Pretreatment

1. Lift station. Consideration should be given to replacing the three existing submersible pumps with three new Gorman Rupp T-series, self-priming pumps. These pumps have excellent solids-passing capability and are easier to maintain since they are not submersible. This pump change would not normally impact effluent quality, but reduced maintenance would offer the operators more time for operation and observation of the remaining pretreatment facilities.

Regardless of the lift pumps utilized, the three discharge lines from these pumps to the rotating screen should be replaced with one common forcemain. This will eliminate the problems with trying to regulate the feeding of chemicals into each line.

2. Chemical Feed System. The existing chemical feed system was installed as a temporary system, nearly a year ago by reusing existing facilities and installing some makeshift provisions to pilot test the acid/ferric chloride chemical pretreatment scheme. Now that this chemical feed scheme has proven successful, the chemical feed system should be systematically laid out and permanently hard wired and hard piped. As part of this permanent design, the adequacy of the existing chemical metering pumps should be evaluated.

3. Operation and Maintenance. Written operation and maintenance instructions should be developed for the entire pretreatment system from the primary screens through the effluent sampling and metering station. In general, these instructions should developed as simple itemized lists for each piece or pieces of equipment or system. These lists should be laminated and mounted near the relevant equipment with a master copy kept on file.

Currently when the chemical feed system becomes upset, the operators call CESCO, Inc. to come to the plant to correct the problems. Fortunately CESCO, Inc., located in Bellingham, is normally able to quickly respond to this call for help. Nevertheless a written "decision tree", or other program, needs to be developed so DVF operating personnel can diagnose and correct problems.

4. Dissolved Air Flotation System. The existing DAF tank is unusual in that it is equipped with neither a mechanical surface skimmer nor bottom solids removal provisions. Although it produces good effluent quality, consideration should be given to equipping this tank with a chain and flight mechanism as a positive means of sweeping floating material to the paddlewheel for removal. This will eliminate the periodic need for the operator to manually rake the skimmings to the paddlewheel.

DAF tank should be drained and cleaned each weekend.

The overflow weir at the effluent end of the tank is only about half of the width of the tank. During the peak flow period when the carcass chiller is emptied, the increased water depth over this constricted overflow weir causes water to flow into the skimmings trough. To minimize the increase in water depth over the weir and prevent water entering the skimmings trough, the effluent overflow weir should be extended to span as much of DAF tank width as possible.

Lighting for most of the pretreatment facilities is good at night, but the effluent weir is in the shadows. Since it is necessary to observe this area to visually determine the adequacy of the chemical pretreatment, a new light should be installed, or an existing yard light relocated, to illuminate this area. Consideration might also be given to installing a turbidimeter to continuously monitor the turbidity of the effluent and sound an alarm if it reaches a preset level. This has proven successful in monitoring effluent quality at other poultry plants.

Since flotation in the DAF tank is dependent on the recycle pressurization pump, a second pump should be available.

## CONCLUSIONS

Based on a review of in-plant waste minimization, recycle/reuse, and wastewater pretreatment practices, Draper Valley Farms is currently meeting AKART requirements with their discharge to the City of Mount Vernon. There are a few in-plant waste minimization practices that should be considered, although they would only result in minor amounts of flow reduction. Recycle/reuse of wastewater by Draper Valley is 'state of the art". There are several pretreatment improvements that should be considered or implemented. These improvements would not appreciably improve effluent quality, but may improve the consistency of maintaining these good results. Draper Valley Farms, Inc. has evaluated the potential improvements previously sited and comments have been included as Appendix E.

# 7. EXISTING WASTEWATER TREATMENT PLANT

## SYSTEM HISTORY

The City of Mount Vernon Wastewater Treatment Plant (WWTP) was originally constructed in 1948 and consisted of primary treatment, disinfection, and anaerobic digestion. In 1972, the WWTP was upgraded to secondary treatment with an oxidation tower (biofilter). In 1989, the secondary treatment was converted to an activated sludge process and the biofilter process was taken out of service.

## TOTAL MAXIMUM DAILY LOAD

The Department of Ecology has established a Total Maximum Daily Load (TMDL) for the Skagit River to ensure that water quality standards will not be impaired as projected growth occurs. The TMDL exists for both dissolved oxygen (DO) and fecal coliform. It is applied during a critical period and allocates loads to each of the contributing parties. The City of Mount Vernon's wastewater treatment plant is an entity that has a TMDL load allocation for both DO and fecal coliform during a defined critical period.

The TMDL for dissolved oxygen governs the oxygen demanding substances that can be added to the Skagit River. In particular, it defines loadings of carbonaceous 5-day biochemical oxygen demand ($CBOD_5$) and ammonia ($NH_3$) that can be discharged to the river. The $CBOD_5$ loading can be exchanged with the ammonia loading. The critical period for the DO TMDL is July through October, and the TMDL limits will be imposed during low flow season, defined as July 1 through November 15. The waste load allocations (WLA) for Mount Vernon are 1,902 lbs/day of $CBOD_5$ and 1,188 lbs/day of $NH_3$-N (alternate WLA are 2,712 lbs/day of $CBOD_5$ and 678 lbs/day of $NH_3$). WLA are derived as acute limits and interpreted as daily maximum or weekly limit. $CBOD_5$ can measured as $BOD_5$ with a site specific conversion factor (a conversion factor of 1.125 is used to estimate $BOD_5$). Table 7-1 summarizes the current TMDL limits for DO for Mount Vernon. If the minimum flow in the river is maintained above the required 6,000 cfs, the daily and weekly TMDL limits may not apply.

Table 7-1

| Dissolved Oxygen Total Maximum Daily Load for Mount Vernon for the Skagit River | | |
|---|---|---|
| Parameter[1] | Average Monthly Limit (lb/day) [3] | Maximum Daily ($NH_3$)or Weekly (BOD) Limit (lb/day) [4] |
| CBOD | 1,407 | 1,902 |
| BOD[2] | 1,583 | 2,140 |
| Ammonia as N | 922 | 1,188 |

1. BOD can be exchanged for ammonia, but the oxygen assimilative capacity provided to Mount Vernon must be maintained.
2. BOD is calculated for CBOD based on a ratio of 1.125.
3. Monthly Average Limits will apply from July through October.
4. Maximum Daily and Weekly Limits will apply when the Skagit River's flow rate falls below 6,000 cfs, measured at USGS gauging station number 12200500, at the highway 99 bridge, upstream of Mount Vernon.

The TMDL for fecal coliform governs the fecal coliform loading to the Skagit River. The critical period for the fecal coliform TMDL is year-round, and the TMDL limits will be imposed during both low and high flow seasons. The waste load allocations (WLA) for Mount Vernon is given as a fecal coliform concentration (rather than a loading) and is equal to the NPDES technology-based permit limits (monthly average of 200 cfu/100 mL).

## NPDES Permit

A meeting was held with City Staff and representatives of the DOE, on January 9, 2001, to discuss the updated NPDES permit. Minutes of this meeting are included in Appendix F. Department of Ecology has issued a draft NPDES Permit to the City of Mount Vernon. The final permit was issued September 4, 2001 and is included in Appendix G. The new permit will addresses CSOs, TMDLs, and WWTP issues. In addition, the City is required to perform toxicity testing.

The new permit is effective October 1, 2001 and expires on June 30, 2003. The effluent limits specified in the permit are listed in Table 7-2 and Table 7-3.

Table 7-2

| NPDES Permit Effluent Limits for Conventional Pollutants for the Mount Vernon WWTP | | |
|---|---|---|
| **Parameter** | **Monthly Average** | **Weekly Average** |
| 5-day Biochemical Oxygen Demand (BOD) | 30 mg/L<br>1401 lbs/day | 45 mg/L<br>2102 lbs/day |
| Total Suspended Solids (TSS) | 30 mg/L<br>1401 lbs/day | 45 mg/L<br>2102 lbs/day |
| Fecal Coliform Bacteria | 200/100mL | 400/100mL |
| PH[1] | Within the range of 6.0 to 9.0 | |

1. Interim limit is in affect for the duration of the NPDES, after which time a new limit of: within the range of 6.6 to 9.0 will apply.

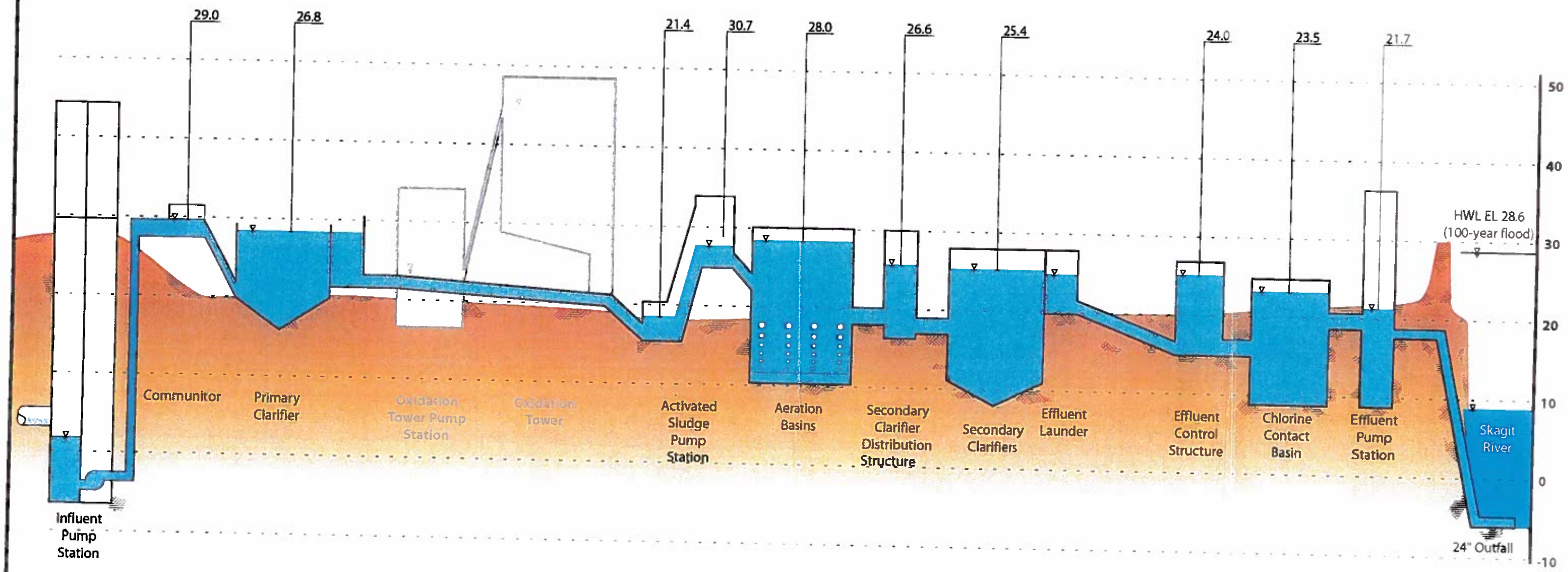
Table 7-3

| NPDES Permit Effluent Limits for Chemical Pollutants for the Mount Vernon WWTP | | |
|---|---|---|
| **Parameter** | **Average Monthly Limit** | **Maximum Daily Limit** |
| Total Residual Chlorine | 50 $\mu$g/L<br>2.21 lbs/day | 100 $\mu$g/L |
| Ammonia-Nitrogen | 31 mg/L<br>1448 lbs/day | 41 mg/L |
| Copper[1] | 21.3 $\mu$g/L<br>1.0 lbs/day | 35 $\mu$g/L |
| Zinc | 88.4 $\mu$g/L<br>4.13 lbs/day | 177.4 $\mu$g/L |

1. Interim limit is in affect for the duration of the draft NPDES, after which time new limits of: Average Day: 9.4 $\mu$g/L, 0.44 lbs/day and Maximum Day 16.6 $\mu$g/L will apply.

## HYDRAULIC PROFILE

The existing WWTP liquid stream processes consists of an influent pump station, screening equipment, primary clarifier, activated sludge pump station, aeration basins, secondary clarifiers, chlorine mixing chamber, chlorine contact basin, and effluent pump station. The hydraulic profile for 12.0 mgd flow (current peak hour capacity) through the existing WWTP is presented in Figure 7-1. The oxidation tower pump station and oxidation tower have been replaced by the activated sludge process and are not currently utilized. Flows from the primary clarifier flow by gravity to the activated sludge pump station.




**Legend**

Existing Unit Processes
Existing Equipment, Not Utilized

29.00 Water Surface Elevation Peak Hour Flow = 12.0 mgd

HDR Engineering, Inc

| | |
|---|---|
| Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE | Date: FEBRUARY 2003 |
| Sheet Title: EXISTING HYDRAULIC PROFILE | Figure No: 7-1 |

## INFLUENT PUMP STATION

The WWTP is primarily served by an influent pump station, which receives flows from the Hazel Street interceptor (42-inch, 24 mgd gravity capacity). The influent sewer enters the pump station approximately 25 feet below grade. The existing pump station is a caisson construction, consisting of a wet well - dry well configuration. A mechanically-cleaned vertical bar screen (1.0-inch spacing) removes large debris from the influent wastewater. A manual bar screen (1.0-inch spacing) is available as backup to the mechanically-cleaned unit. Flows discharge to the existing comminutor through a 20-inch force main. The pumping units consist of four variable-speed, 40-hp pumps. The pump station has a firm pumping capacity of 10.8 mgd.

## WEST MOUNT VERNON PUMP STATION

The WWTP also receives flows from the West Mount Vernon Pump Station. The pump station capacity is 1.2 mgd. Flows enter the WWTP through a 12-inch force main and discharge at the head of the existing comminutor.

## HEADWORKS

The headworks of the existing WWTP consists of comminution and de-gritting primary sludge. The comminutors are located downstream of both pump stations, and immediately upstream of the primary clarifier. Grit removal is located downstream of the primary clarifier, where primary sludge is de-gritted.

### Comminutor

Comminution at the WWTP is performed by two comminutors, with a capacity of 12.0 mgd.

### Grit Removal

The WWTP currently degrits primary sludge. Primary sludge is removed from the primary clarifiers and sent through an existing grit separator. The grit is then stored until it is removed for disposal.

### Disposal

Screenings and grit are transported to a county landfill for final disposal.

## PRIMARY CLARIFIER

The existing primary clarifier is an 80-foot-diameter circular tank with a surface area of approximately 5,000 sf and a sidewater depth of 10-foot. It is center well fed with a peripheral effluent launder. It has a peak hour design capacity of 12.0 mgd at a surface loading rate of 2,400 gpd/sf. The water surface elevation (at 12.0 mgd) is 26.81 feet. A parallel unit process does not currently exist for the primary clarifier for backup service.

## OXIDATION TOWER AND OXIDATION TOWER PUMP STATION

The oxidation tower pump station consists of two (2) 75 hp pumps. The oxidation tower is a 48-FT long, 40-FT wide, and 16-FT deep tower filled with redwood media. Primary effluent was pumped to the top of the tower and trickled down the redwood media. Biofilm on the media removed the organic pollutants from the primary effluent with oxygen provided by natural aeration. This system was taken out of service when the previous plant upgrade was completed, which included aeration basins and appurtenances for the activated sludge process. As a part of this study an analysis was completed to see if it would be cost effective to incorporate this existing plant componenet into a future plant upgrade. It was concluded that this was not cost effective to incorporate this existing plant component into a future plant upgrade. It was also concluded that this was not a cost effective alternative for providing increased treatment capacity.

The oxidation tower should be removed to provide a location for additional required equipment. The costs for removal of the structure will be incorporated into the costs associated with the new equipment that will be placed at this location.

## ACTIVATED SLUDGE PROCESS

### Activated Sludge Pump Station

The activated sludge pump station conveys primary effluent to the aeration basins. The pump station consists of three screw-lift pumps. Each has a capacity of 8.0 mgd. Two are designated for forward flow (16.0 mgd) and one is designated for return activated sludge (RAS) flow (8.0 mgd).

### Aeration Basins

Aeration Basins Nos. 1-3 each have a volume of 0.33 MG, for a total aeration basin volume of 1.0 MG. Aeration Basin No. 4 which has a volume of 0.47 MG, also is available for use as an aeration basin, but will require modifications to the inlet and outlet piping. However, it is currently used as a WAS holding tank, allowing 24-hour wasting and flexibility in operating the dissolved air floatation thickener.

## INFLUENT PUMP STATION

The WWTP is primarily served by an influent pump station, which receives flows from the Hazel Street interceptor (42-inch, 24 mgd gravity capacity). The influent sewer enters the pump station approximately 25 feet below grade. The existing pump station is a caisson construction, consisting of a wet well - dry well configuration. A mechanically-cleaned vertical bar screen (1.0-inch spacing) removes large debris from the influent wastewater. A manual bar screen (1.0-inch spacing) is available as backup to the mechanically-cleaned unit. Flows discharge to the existing comminutor through a 20-inch force main. The pumping units consist of four variable-speed, 40-hp pumps. The pump station has a firm pumping capacity of 10.8 mgd.

## WEST MOUNT VERNON PUMP STATION

The WWTP also receives flows from the West Mount Vernon Pump Station. The pump station capacity is 1.2 mgd. Flows enter the WWTP through a 12-inch force main and discharge at the head of the existing comminutor.

## HEADWORKS

The headworks of the existing WWTP consists of a comminutor and de-gritting primary sludge. The comminutor is located downstream of both pump stations, and immediately upstream of the primary clarifiers. Grit removal is located downstream of the primary clarifiers, where primary sludge is de-gritted.

### Comminutor

Comminution at the WWTP is performed by two comminutors, with a capacity of 12.0 mgd.

### Grit Removal

The WWTP currently degrits primary sludge. Primary sludge is removed from the primary clarifiers and sent through an existing grit separator. The grit is then stored until it is removed for disposal.

### Disposal

Screenings and grit are transported to a county landfill for final disposal.

## PRIMARY CLARIFIER

The existing primary clarifier is an 80-foot-diameter circular tank with a surface area of approximately 5,000 sf and a sidewater depth of 10-foot. It is center well fed with a peripheral effluent launder. It has a peak hour design capacity of 12.0 mgd at a surface loading rate of 2,400 gpd/sf. The water surface elevation (at 12.0 mgd) is 26.81 feet. A parallel unit process does not currently exist for the primary clarifier for backup service.

## OXIDATION TOWER AND OXIDATION TOWER PUMP STATION

The oxidation tower pump station consists of two (2) 75 hp pumps. The oxidation tower is a 48-FT long, 40-FT wide, and 1 6-FT deep tower filled with redwood media. Primary effluent was pumped to the top of the tower and trickled down the redwood media. Biofilm on the media removed the organic pollutants from the primary effluent with oxygen provided by natural aeration. This system was taken out of service when the previous plant upgrade was completed, which included aeration basins and appurtenances for the activated sludge process. As a part of this study an analysis was completed to see if it would be cost effective to incorporate this existing plant componenet into a future plant upgrade. It was concluded that this was not cost effective to incorporate this existing plant componenet into a future plant upgrade. It was also concluded that this was not a cost effective alternative for providing increased treatment capacity.

The oxidation tower should be removed to provide a location for additional required equipment. The costs for removal of the structure will be incorporated into the costs associated with the new equipment that will be placed at this location.

## ACTIVATED SLUDGE PROCESS

### Activated Sludge Pump Station

The activated sludge pump station conveys primary effluent to the aeration basins. The pump station consists of three screw-lift pumps. Each has a capacity of 8.0 mgd. Two are designated for forward flow (1 6.0 mgd) and one is designated for return activated sludge (RAS) flow (8.0 mgd).

### Aeration Basins

Aeration Basins Nos. 1-3 each have a volume of 0.33 MG, for a total aeration basin volume of 1.0 MG. Aeration Basin No. 4 which has a volume of 0.47 MG, also is available for use as an aeration basin, but will require modifications to the inlet and outlet piping. However, it is currently used as a WAS holding tank, allowing 24-hour wasting and flexibility in operating the dissolved air floatation thickener.

### Aeration Blowers

There are four existing Lamson centrifugal blowers, each rated at 4,100 scfm at 8.5 psi. The maximum air supply with one blower out of service is 12,300 scfm.

### Secondary Clarifiers

Secondary clarification is performed with two 85-foot diameter secondary clarifiers. Secondary Clarifier No. 1 has an 11-foot sidewater depth and a peripheral feed. Secondary Clarifier No. 2 has a 15-foot sidewater depth and a more conventional center well feed.

## DISINFECTION

The existing disinfection system consists of gaseous chlorine injection followed by a chlorine contact basin. The chlorination equipment, two chlorinators, each have a capacity range of 100 to 2,000 ppd. The chlorine contact basin has a volume of 184,000 gallons, and a contact time of 66 minutes at 4.0 mgd and 22 minutes at 12.0 mgd.

## EFFLUENT PUMP STATION

The effluent pump station consists of three 40 hp pumps, each with a capacity of 7.2 mgd. The firm pumping capacity of the station is 12.0 mgd.

The effluent pump station is only necessary when the river's water surface elevation (WSEL) increases due to flood conditions. Under normal conditions (WSEL of 9.20 feet), effluent flows by gravity to the Skagit River. The 100-year flood WSEL is 28.60 feet (based on 1987 WWTP improvement contract documents).

## OUTFALL

The existing outfall is a 24-inch diameter, open-ended, ductile iron pipe. The pipe terminates adjacent to the treatment plant at River Mile 10.7 on a well armored slope of the Skagit River. It is located within a small depression in the riverbank. This depression creates an eddy that visibly traps effluent near the shoreline.

## SOLIDS TREATMENT

### Gravity Thickener

The gravity thickener is designated for primary sludge thickening, before discharge to the anaerobic digester. The tank is 22-foot diameter and has a 10-foot sidewater depth.

### Dissolved Air Floatation Thickener (DAFT)

The existing DAFT is a 40-foot diameter tank with an 11-foot sidewater depth. WAS is currently stored in Aeration Basin No. 4 before discharge to the DAFT. Polymer is added to the WAS at the DAFT unit. Thickened WAS is fed to the anaerobic digester.

### Anaerobic Digester

The existing anaerobic digester is a 60-foot digester with a 34-foot sidewater depth. It has a volume of 103,200 cf. The digester utilizes a gas mixing system and is provided with a floating cover for gas storage.

### Solids Dewatering

Dewatering is accomplished with two 2-meter belt filter presses. Each unit has a capacity of 1,100 pph, for a combined capacity of 2,200 pph. The 75 foot diameter circular tank (original primary clarifier) is used as a holding tank for the biosolids transferred from the primary digester, prior to dewatering via belt filter press.

## ODOR CONTROL

To control odors, the City currently doses the liquid stream with chlorine, both in the collection system and at the WWTP. Odors from the solids processes at the WWTP are not treated. The City currently owns the majority of the property around the WWTP, providing an additional buffer zone for dispersing odors.

## FACILITIES

### Operations Building

The existing operations building consists of two offices, men's and women's lockers, a lunch room, a control room, and a laboratory. The control room has a floor area of approximately 175 sf and contains control panels, computers, and printers. The laboratory has a floor area of approximately 420 sf and includes one fume hood, three sinks, one balance table, one refrigerator, and one incubator.

### Shop/Garage

The existing shop/garage consists of four areas:

- 375 sf shop area;
- 70 sf wash room;
- 60 sf storage area; and
- 2,000 sf garage area, divided into 5 bays.

# 8. WASTEWATER TREATMENT PLANT ANALYSIS

This chapter analyzes the capacity of the existing treatment system and predicts facilities required to meet future flows and loads as presented in Chapter 3, for years 2010 and 2020.

## 2010 AND 2020 TREATMENT REQUIREMENTS

The Total Maximum Daily Load (TMDL) for the Skagit River in conjunction with the NPDES permit limits determine the concentrations and loadings that can be discharged during the low flow season. The total loadings are based on a sum of loads from the WWTP outfall and the CSO outfalls. These maximum TMDL limits are listed in Table 8-1

**Table 8-1**

| Skagit River BOD and $NH_3$ TMDL Limits | | |
|---|---|---|
| **Parameter** | **Maximum Daily ($NH_3$) or Weekly (BOD) TMDL Limit (lb/day)** | **Average Monthly TMDL Limit (lb/day)** |
| BOD | 2140 | 1583 |
| $NH_3$ | 1188 | 922 |

The existing effluent flows from the WWTP for 1998 during the TMDL season (July through November) were:

- BOD: Average monthly concentrations from 12 to 20 mg/L, with a maximum weekly concentration of 26 mg/L; and
- $NH_3$: Average monthly values ranged from 18 to 31 mg/L, with maximum day ammonia concentrations ranging from 22.7 to 43.9 mg/L (July through October of 1999 and 2000).

Future effluent BOD and $NH_3$ loadings from the WWTP and CSO flows were estimated to determine if TMDL limits would be met. CSO loadings were determined from the largest CSO loading during the TMDL season, which occurred during the August 18, 2000 storm event. Table 8-2 summarizes the projected effluent and CSO loadings to the Skagit River during the TMDL season.

**Table 8-2**

| Estimated $BOD_5$ and $NH_3$ Loadings to the Skagit River During the Time Average Monthly TMDL Limits Apply (July - October). | | | | |
|---|---|---|---|---|
| **Year and Location** | **Weekly $BOD_5$ Load (lb/day)** | **Average Monthly $BOD_5$ Load (lb/day)** | **Maximum Daily $NH_3$ Load (lb/day)** | **Average Monthly $NH_3$ Load (lb/day)** |
| 2000 WWTP | 752[1] | 585[2] | 919[3] | 666[4] |
| CSO (August 18, 2000) | 11 | 11 | 0.3 | 0.3 |
| Total 2000 Loading | 763 | 596 | 919 | 666 |
| | | | | |
| 2010 WWTP[5] | 1,128 | 878 | 1,379 | 999 |
| CSO (August 18, 2000) | 11 | 11 | 0.3 | 0.3 |
| Total Estimated 2010 Loading | 1,139 | 889 | 1,379 | 999 |
| | | | | |
| 2020 WWTP[6] | 1,379 | 1,073 | 1,685 | 1,222 |
| CSO (August 18, 2000) | 11 | 11 | 0.3 | 0.3 |
| Total Estimated 2020 Loading | 1,390 | 1,084 | 1,685 | 1,222 |
| | | | | |
| TMDL Limit | 2,140 | 1,583 | 1,188 | 922 |
| Last Year in Compliance | -[7] | -[7] | 2005[8] | 2007[8] |

1. Maximum weekly BOD load from October 1999
2. Average monthly BOD load from October 1999
3. Maximum day ammonia load from August 2000
4. Average monthly ammonia load from August 2000
5. Based on the ratio of 2000 ADMM to predicted 2010 ADMM
6. Based on the ratio of 2000 ADMM to predicted 2020 ADMM
7. Estimated loadings will be in compliance through 2020
8. Estimated loadings will exceed current TMDL limits. TMDL limits are not expected to change with future permits or studies.

Based on the existing effluent characteristics and the TMDL limits, the WWTP will be required to nitrify, by the summer of 2006, in order to meet future NPDES permit limits. This estimation of when nitrification will be required may vary dependant upon effluent flow rates, WWTP performance, and actual daily ammonia loadings.

## INFLUENT PUMP STATION

### Pumping Capacity

The firm pumping capacity of the Influent Pump Station is 10.8 mgd. The projected 2010 and 2020 peak hour flows are 14.9 and 18.3 mgd respectively.

City staff have noted problems with the existing pump station configuration. Pump Nos. 2 and 3 are affected by the discharge of the influent wastewater adjacent to the suction inlets for the pumps. The pumps can become air-bound and this can limit discharge capacity.

The maximum capacity of the 42-inch diameter interceptor supplying the wet well is 24 mgd. The pump station and force main should be upgraded to a firm pumping capacity of 24 mgd to maximize the conveyance of wastewater flows (both sanitary and combined sewer flows) to the WWTP. This is consistent with the recommended long term CSO improvements (Alternative 2C) identified in Chapter 4.

### Screening

Coarse screening is currently provided in the Influent Pump Station by mechanically-cleaned bar screens with manually-cleaned bar screens as a backup unit.

The plant cperating staff has expressed concerns over the operation and maintenance of the manually cleaned bar screen. It is located upstream of the pump station wet well, approximately 24-feet below grade. Screenings must be conveyed from the screen to a location approximately 4-feet above grade.

## HEADWORKS

The existing headworks facility consists of a comminution and de-gritting of primary sludge. The City has noted excessive wear on the WWTP process equipment due to grit and debris that could be removed by fine screens and grit chambers.

### Comminutor

The purpose of a comminutor is to shred material in the flow stream. A problem associated with this process is that the material often reconstitutes later in the flow stream. A better method is to remove solid materials with fine screens, further process this material in a solids washer to remove organic material, and remove the non-organic material from the flow stream.

### Screening

Fine screening is recommended as a replacement to the comminutor. These screens would have three-eighths-inch openings and be mechanically cleaned. They would be placed downstream of all influent flows (WWTP influent pump station and West Mount Vernon Pump Station), and upstream of the recommended grit removal equipment. The parameters used to size fine screens are the peak hour flow.

### Grit Removal

The current grit removal system removes grit from the primary sludge. The trend in current grit removal technology is to remove the grit in the flow stream prior to primary clarification. This can be accomplished by settling grit, via centrifugal forces, in a variety of geometrical chambers, circular, square, or rectangular. Removal of grit prior to primary sedimentation allows for flexibility with the primary clarifiers, such as thickening of the primary sludge in the clarifier.

### Disposal

Screening (both course and fine) processes can be expected to produce five to ten cubic feet of screening per million gallons of wastewater treated. The volume of screenings to be landfilled can be reduced through washing and compacting. The grit removal process can be expected to produce one to three cubic feet of grit per million gallons of wastewater treated. The presence of organic matter in the grit to be landfilled will be reduced through washing. Odors can be a concern for storage of screenings and grit until final disposal.

## CAPACITY ANALYSIS

A capacity analysis was completed which evaluated the primary treatment, secondary treatment, and solids handling facilities. A mass balance model of the entire treatment plant was constructed using HDR's ENVision program. This model incorporates flows and pollutant loads from both influent and internal recycle streams. Process loading conditions derived from the mass balance output were calibrated to standard and historical plant performance data.

Table 8-3 provides a summary of the capacity analysis. The first three columns summarize the existing facilities, volumes and dimensions. The next four columns list the capacity evaluation criteria, the flow rate that each criterion applies to, and the reference. The two columns titled "Value with BOD removal" and "Value with Nitrification" present the predicted process variables from the ENVision model if the 2020 future flows were directed through the existing facilities. The columns titled "Capacity of existing facilities-BOD removal" and "Capacity of existing facilities-Nitrification" list the flow capacities (either maximum month, maximum day or peak hour as indicated in the capacity flow column) for the listed process with BOD removal and with nitrification. The last two columns of the table list the additional facilities that would be required to meet the criteria shown. The largest value under each

process is shown in bold. The value in bold will determine the sizing for design of new facilities. Model data summary sheets are included as Appendix H.

The ENVision model was run for each flow and loading condition shown in Table 3-8 and Table 3-9. From the model output, the capacity of the existing facilities was calculated, and new facilities were proposed. For example, the existing primary clarifier was run at maximum month flow conditions (9.9 mgd) the overflow rate was 2,100 gpd/sf as shown in the first row of Table 8-2. Because the criteria listed is 1,000 gpd/sf, the capacity of the existing primary clarifiers is a maximum month flow of [(1,000/2,100) x 9.9] 5 mgd. The existing primary clarifier is 5,000 sf in area. To meet the 2020 maximum month flow condition, a total of 9,900 square feet are required. Therefore, (9,900-5,000) 4,900 sf must be added. Capacities and required volumes and areas of other processes were computed in a similar fashion.

For BOD removal, the model was run at a 4-day SRT, the average SRT of the existing facility. For nitrification analysis, the model was run at a 10-day SRT to ensure full nitrification.

## Table 8-3– 2020 Process Capacity Analysis with ENVision Model

| Process | Existing Facility Description | Size or Capacity | Criteria Flow | Parameter | Capacity Criteria | Reference | Value with BOD removal[1] | Value with Nitrification[1] | Capacity of existing facilities BOD removal | Capacity of existing facilities Nitrification | Additional Facilities Req'd BOD removal | Additional Facilities Req'd Nitrification |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Primary Clarifier | 1-primary clarifier | 80 ft diameter | MM | OFR | 1,000 gpd/sf | DOE Standard | 2,100 gpd/sf | Same as BOD | 5.0 MGD | Same as BOD | **5,100 sf** | Same as BOD |
| | | 10 ft side water depth | PH | OFR | 2,500 gpd/sf | DOE Standard | 3,800 gpd/sf | Same as BOD | 12.5 MGD | Same as BOD | 2,400 sf | Same as BOD |
| | | 5,000 sf | | | | | | | | | | |
| | | 0.4 MG | MM | HRT | 2.5 hr | DOE Standard | 0.9 hr | Same as BOD | -- | Same as BOD | -- | Same as BOD |
| Aeration Basins | 3-plug flow aeration basins | 61 ft length, | | | | | | | | | | |
| | | 42 ft width, | MM | MLSS | 2,500 mg/L | Stress testing | 3,000 mg/L | 6,400 mg/L | 8.2 MGD | 3.9 MGD | 0.2 MG | 1.6 MG |
| | | 17.5 ft SWD | | | | | | | | | | |
| | | 0.33 MG each, | MD | MLSS | 2,700 mg/L | Stress testing | 2,800 mg/L | 7,200 mg/L | 11.0 MGD | 5.2 MGD | -- | **1.7 MG** |
| | | 1.0 MG total | | | | | | | | | | |
| | 1-aeration basin (WAS storage) | 61 ft length | | | | | | | | | | |
| | | 60 ft width | | | | | | | | | | |
| | | 17.5 ft SWD | | | | | | | | | | |
| | | 0.5 MG total | | | | | | | | | | |
| Aeration System—Diffusers | 9-inch diameter coarse bubble diffusers | | MM | OUR | 32 mg/L-hr[2] | HDR Standard | 38 mg/L-hr | 73 mg/L-hr | 8.9 MGD | 4.6 MGD | 0.4 | 1.5 MG |
| | | | MD | OUR | 36 mg/L-hr[2] | HDR Standard | 45 mg/L-hr | 80 mg/L-hr | 8.4 MGD | 4.7 MGD | 0.4 | 1.2 MG |
| | | | PH | OUR | 54 mg/L-hr[2] | HDR Standard | 56 mg/L-hr | 113 mg/L-hr | 18.1 MGD | 9.0 MGD | 0.1 | 1.1 MG |
| Aeration System—Blowers | 4-centrifugal | 4,100 scfm each | MD | SCFM | 12,300 | None | 5,900 scfm | 12,800 scfm | 24.0 MGD | 13.4 MGD | None | 500 scfm |
| | | 12,300 scfm with 1 out of service | PH | SCFM | 12,300 | None | 9,000 scfm | 16,600 scfm | 25.0 MGD | 13.6 MGD | None | **4,300 scfm** |
| Secondary Clarifiers | 2-secondary claritiers | 85-ft diameter | MD | HRT | <2 hr | HDR Standard | 1.5 hr | 1.5 hr | -- | Same as BOD | -- | Same as BOD |
| | | 1-11 ft SWD | PH | HRT | | | | | | | | |
| | | 1-15 ft SWD | PH | OFR | 900 gpd/sf | DOEStd 1,200 | 1,600 gpd/sf | 1,600 gpd/sf | 10.2 MGD | Same as BOD | 9,000 **sf** | Same as BOD |
| | | 5,700 sf each | | | | | | | | | | |
| | | 1-0.47 MGD | MM | SLR | 25 lb/d-sf[2] | DOE Standard | 28 lb/d-sf | 28 lb/d-sf | 9.0 MGD | Same as BOD | 1,100 sf | Same as BOD |
| | | 1-0.64 MGD | MD | SLR | 40 lb/d-sf[2] | DOE Standard | 32 lb/d-sf | 38 lb/d-sf | 14.5 MGD | Same as BOD | -- | Same as BOD |
| | | | PH | SLR | 40 lb/d-sf[2] | DOE Standard | 51 lb/d-sf | 51 lb/d-sf | 14.5 MGD | Same as BOD | 3,000 sf | Same as BOD |
| Gravity Thickener | 1-thickener | 22-ft diameter | MM | OFR | 700 gpd/sf | DOE Standard | 261 gpd/sf | 353 gpd/sf | 28.2 MGD | 19.6 MGD | None | None |
| | | 10-ft SWD | | | | | | | | | | |
| DAF Thickener | 1- DAF Thickener | 40-ft diameter | MM | SLR | 2.5 lb/hr-sf | DOE Standard | 4.0 | 3.5 | -- | 5.6 MGD | **750 sf** | **500 sf** |
| | | 11 ft SWD | | | | | | | | | | |
| | | 1,260 sf | | | | | | | | | | |
| Anaerobic Digester | 1-anaerobic digester | 60 ft diameter, | MM | SRT | 15 days | EPA Standard | 28 d | 33 d | 23 MGD | 10 MGD | None | None |
| | | 34 ft SWD, | MM | SLR | 140 lbVSS/kcf-d | WEF MOP8 | 80 lbVSS/kcf-d | 70 lbVSS/kcf-d | 18 MGD | 13 MGD | None | None |
| | | 103,400 cf (0.8 MG) | | | | | | | | | | |

*[1] Values in this column were determined using the ENVision model calibrated to the existing facility.*
*[2] These values assume conversion to fine bubble diffusers.*

DAF-dissolved air flotation
DOE-Department of Ecology
kcf-1000 cubic feet
HRT-hydraulic retention time
MD-maximum day
MG-million gallons
MGD-million gallons per day
MLSS-mixed liquor suspended solids
MM-maximum month average day
OFR-overflow rate
OUR-oxygen uptake rate
PH-peak hour
SCFM-standard cubic feet per minute
SLR-solids loading rate
SRT-solids retention time
SWD-sidewater depth
VSS-volatile suspended solids
WEF-Water Environment Federation

### Primary Clarifiers

Primary clarification was evaluated based on both hydraulic residence time (HRT) and overflow rate. The DOE standard for average day maximum month overflow rate is 800-1,200 gpd/sf. A value of 1,000 gpd/sf was used as the design primary clarifier overflow rate (OFR). Similarly, the DOE standard for peak hour OFR is 2000-3000 gpd/sf and 2,500 gpd/sf was used as the design criterion.

DOE recommends an HRT of less than 2.5 hours for primary clarifiers under average day maximum month loading conditions to prevent septic conditions in the clarifier.

The additional primary clarifier area required to meet the peak hour OFR requirement is more than the additional area required to meet the maximum month requirement. It is recommended that the total 2010 primary clarifier area be a minimum of 10,100 sf and the total 2020 primary clarifier area be a minimum of 10,100 sf.

### Aeration Basins

Aeration basin volume was evaluated based on MLSS concentrations and oxygen uptake rates. The October 1995 Plant Evaluation presented data on secondary clarifier stress testing. It showed that the deeper of the two secondary clarifiers (Secondary Clarifier No. 2) could handle MLSS concentrations above 3,600 mg/L. Data on MLSS capacity of the shallower clarifier (Secondary Clarifier No. 1) was not presented. The capacity criteria for MLSS are 2,500 mg/L under maximum month loading conditions and 2,700 mg/L under maximum day loading conditions.

Aeration volume was also evaluated based on oxygen uptake rates. Typical oxygen uptake rates for aeration basins with fine bubble diffusers are 32, 36, and 54 mg/L-hr for maximum month, maximum day and peak hour conditions, respectively. The volumes required to meet oxygen uptake rate requirements were all equal to or lower than those required to meet MLSS criteria, therefore the MLSS criteria will be used to determine basin size.

If BOD removal is the treatment goal (no nitrification), then an additional 0.2 MG of aeration volume would be required to meet the future flow and loading conditions. If the existing Aeration Basin No. 4 (0.5 MG) was converted from a WAS holding tank to an aeration facility, no new basin construction would be required, but the coarse bubble diffusers would have to be changed to fine bubble diffusers.

If nitrification is the treatment goal, then an additional 1.7 MG of aeration volume would be required to meet the future flows and loads. Aeration basin 4 could be converted reducing the required aeration basin volume for construction to 1.2 MG. Based on the January 9, 2001, meeting with the City and representatives of DOE, it appears the NPDES permit currently being prepared will not require nitrification, but the future permits could contain these requirements.

If total nitrogen removal were desired (denitrification), then the total aeration volume would increase by approximately 30%. For a total aeration volume of 2.7 MG an additional 0.9 MG may be required for denitrification. Denitrification would lower aeration requirements

and increase alkalinity to the downstream processes. At this time, a requirement for denitrification is not anticipated in the next ten years (two permit cycles).

### Aeration Blowers

For BOD removal, a total of 6,800 scfm would be needed to meet 2020 peak hour requirements. There are currently four 200 hp centrifigal blowes each rated at a capacity of approximately 4,100 scfm.each. For nitrification, however, 16,600 scfm would be required under 2020 peak hour loading conditions; 4,300 more than 12,300 available.

An additional blower would be required to meet peak hour loads if a redundant blower were to be maintained during peak hour loading conditions for 2020 loadings and operation in the nitrification mode, however this is very conservative criteria and many plants are designed to provide firm blower capacity for the maximum day loadings and total capacity for the peak hour loading conditions. At this time additional blower capacity is not recommended for the year 2020 improvements.

### Secondary Clarifiers

The secondary clarifiers were evaluated based on HRT, overflow rate, and solids loading rate. The DOE guideline for secondary clarifier overflow rates is 600 to 800 gpd/sf for average day, maximum month conditions. The DOE recommended maximum overflow rate for peak hour conditions is 1,200 gpd/sf. In this case, since the sewer system is a combined sewer system with storage provided by the Central CSO Regulator, the CSO flows can be stored in the regulator and discharged to the treatment plant over an extended period of time. For this reason, the allowable peak hour loading for the secondary clarifiers was reduced to 900 gpd/sf to prevent the washout of solids during extended periods of high flow resulting from storm events. The total surface area required for 2020 is approximately 20,400 sf.

The DOE standard for secondary clarifier solids loading under average day maximum month conditions is up to 25 lb/d-sf. At peak conditions, DOE lists a peak maximum loading rate of 40 lb/d-sf. The clarifier stress testing indicated that Secondary Clarifier No. 2 is capable of handling at least 25 lb/d-sf and probably higher loading rates. Secondary Clarifier No. 1, however, was capable of only 12 lb/d-sf under test conditions. The areas required to meet all solids loading criteria were less than the 7,600 sf required to meet the 900 gpd/sf OFR sizing criteria. If the existing 85 foot diameter peripheral feed secondary clarifier with the 12 foot sidewater depth was eliminated, the additional surface area required for 2020 would be 14,700 square feet.

### Gravity Thickener

DOE recommends 600-800 gpd/sf overflow rate for gravity thickeners. An overflow rate of 700 gpd/sf has been used for this evaluation. Under 2020 future solids loadings, both with and without nitrification the overflow rate is less than 300 gpd/sf and no additional gravity thickening improvements are needed.

If the grit removal is relocated upstream of the primary clarifier, the option of thickening solids within the primary clarifier will also be available. If the grit removal was provided, the gravity thickener would be maintained for backup service.

### Dissolved Air Floatation Thickener

The DOE standard for solids loading rate to a DAFT with polymer addition is up to 2.5 lb/hr-sf. The surface area of the existing unit is 1,250 square feet. Under 2020 future loads, an additional 750 sf would be required if BOD was removed or 500 sf if the plant is operated in the nitrification mode. In either case, an additional unit would be required and should also be provided for redundancy.

### Anaerobic Digester

The EPA 503 regulations recommend a minimum 15-day SRT in anaerobic digesters to meet Class B requirements. Under future flows, with BOD removal only, the SRT would be 33 days and with nitrification the SRT would be 28 days; well above the 15-day requirement. The Water Environment Federation Manual of Practice recommends anaerobic digesters be loaded at a maximum of 140 lb VSS/kcf-d solids loading. Under future flows and loads, the solids loading would be 80 lb VSS/kcf-d and 70 lb VSS/kcf-d with BOD removal and nitrification, respectively; below the maximum loading of 140 lb VSS/kcf-d. According to the ENVision model, additional digester capacity is not anticipated under 2020 flows and loads.

The City reports hydraulic capacity of the digester is presently limited due to grit deposition at the bottom and a scum layer at the top. Assuming a 30% reduction in available volume, the available SRT would be 19 days for the year 2020 loadings. Additionally, there is limited capacity to store solids when the existing primary digester is taken out of service for cleaning. Presently during digester cleaning, Aeration Basin No. 4 is used as an aerobic digester. A redundant unit process should be considered to alleviate the problems associated with storing biosolids while cleaning the existing digester, and to ensure a hydraulic capacity limitation does not exist in the future.

### Solids Dewatering

Solids dewatering is currently performed via two (2) belt filter press. The City operates the presses (based on daily operation of one belt filter press, 1,100 pph) for an average of 2.3 hours per day. Under 2010 flow conditions, the belt filter presses would be required to be operated for 4.2 hours per day. Under 2020 flow conditions, the belt filter presses would be required to be operated for 4.9 hours per day. The existing belt filter presses are adequate and no additional dewatering improvements are needed.

## DISINFECTION

Gaseous chlorine is presently used for disinfection of the effluent, followed by dechlorination with sodium bisulfite. Due to the safety concerns over the storage of one ton gaseous chlorine cylinders, the costs of complying with increasingly stringent hazardous materials regulations governing the storage of gaseous chlorine, and the environmental benefits of ultraviolet (UV) disinfection, the City of Mount Vernon decided to evaluate alternative disinfection methods at the WWTP. UV disinfection alternatives are developed in the following chapter.

If gaseous chlorine is eliminated, there would still be a need for chlorine for housekeeping items such as algae control, odor control, and sludge bulking control. In this case a sodium hypochlorite system could be provided for these needs.

## EFFLUENT PUMP STATION

The existing effluent pump station is not sized to convey 2010 or 2020 peak hour flow rates to the Skagit River. The pump station should be upgraded to maximize conveyance of effluent from the WWTP. The parameter used to size pumps for the Effluent Pump Station is the peak hour flow and the 100-year water surface elevation of the Skagit River.

## OUTFALL

A mixing zone study of the existing WWTP outfall was performed by Cosmopolitan Engineering Group, Inc. in February 2000. This report notes that effluent, when tracked by Rhodamine WT dye, was visibly trapped in a near-shore eddy. Mixing of the effluent and ambient water occurred at the offshore boundary of the eddy. From this analysis, it was determined that modifications to the existing outfall should occur. The flow parameters used to design the outfall are:

| Flow Condition | Criteria |
|---|---|
| • Peak Hour Flow | Hydraulic Capacity |
| • Maximum Day Flow | Acute Mixing Zone Requirements |
| • Average Day Maximum Month Flow | Chronic Mixing Zone Requirements |

The outfall design also is affected by the NPDES permit limits and the water quality criteria of the receiving water body.

Mixing zones as defined by Mount Vernon's NPDES permit:

Chronic Mixing Zone:

- Shall not exceed greater than 300 feet plus the water depth downstream, or 100 feet upstream;
- Shall not utilize greater than 25 percent of the river flow; and
- Shall not occupy greater than 25 percent of the river width.

Acute Mixing Zone:

- Shall not extend beyond 10 percent of the distance to the chronic mixing zone boundary; and
- Shall not utilize greater than 2.5 percent of the river flow.

Water quality standards for toxicants:

| Parameter | Acute Criteria (μg/L) | Chronic Criteria (μg/L) |
|---|---|---|
| Chlorine | 19 | 11 |
| Ammonia-N | 8,314 | 1,877 |
| Copper | 4.61 | 3.47 |
| Mercury | 2.1 | 0.012 |
| Lead | 13.9 | 0.54 |
| Silver | 0.32 | - |
| Zinc | 35.4 | 32.3 |

To comply with the mixing zone and water quality criteria, a new or modified outfall will be required. Prior to construction of this improvement, the City will be required to obtain multiple permits. The following is a preliminary listing of anticipated permits/approvals for outfall modifications:

| Agency/Jurisdiction | Permit/Approval |
|---|---|
| • U.S. Army Corps of Engineers[1] | • Section 10/404 Permit |
| | • Biological Evaluation/Biological Assessment |
| • WA Department of Fish and Wildlife | • Hydraulic Project Approval |
| | • Priority Habitat Review |
| • WA Department of Ecology | • Waste Discharge Permit Review (NPDES)[2] |
| | • Section 401 Water Quality Certification |
| • WA Department of Natural Resources | • Aquatic Use Authorization[3] |
| • City of Mount Vernon | • Shoreline Permit |
| | • Floodplain Review |
| | • Sensitive/Critical Area Review |
| | • SEPA |
| | • Dike Setback Variance |
| | • Fill and Grading Permit |
| • Dike District No. 3 | • Dike District Approval |

1. The U.S. Army Corps of Engineers is now requiring a Biological Evaluation or Biological Assessment for all projects requiring Corps approval. This will trigger consultation with the National Marine Fisheries Service and the U.S. Fish and Wildlife Service. Chinook salmon, bull trout and bald eagle are known to occur in the project vicinity and will mostly likely, after consultation with NMFS/USFWS, be included in the BE/BA.
2. It is anticipated that the existing NPDES permit will require modification or a new NPDES permit may be required.
3. Any project that is located on state-owned aquatic lands will require authorization from the WDNR. The Skagit River at the outfall location is considered state-owned lands.

A detailed examination of the required permits and an estimated schedule for obtaining permits is presented in Appendix I.

## ODOR CONTROL

Chlorine is presently injected in the incoming wastewater flow at Hazel Street and Harrison Street. This has been relatively successful, but requires significant quantities of chlorine. The chlorine is presently supplied from the gaseous chlorination system at the WWTP. Typically, chlorine usage at the plant is:

| Usage | Approximate Chlorine Usage (ppd) |
|---|---|
| Disinfection | 30 |
| Odor Control | 50 to 200 |
| Process Control | 100[1] |
| Maximum Day Usage | 330 |

1. Process control is for filamentous control

In addition to reducing odor potential within the collection and conveyance system, odor control at wastewater treatment facilities often includes treatment of odors in the gaseous phase on site. This includes containment of the gases at the process locations (i.e. covers on tankage where odors occur) or containment of odors within facilities with higher odor (i.e. headworks building). Ventilation is provided to transfer the high odor air to odor treatment units. These can consist of packed tower liquid scrubbers, activated carbon absorption, or biological treatment with compost filters.

After UV disinfection at the WWTP is implemented, gaseous chlorination would eventually be eliminated. Small chlorine requirements for process control would be met with hypochlorite, but meeting high chlorine demands with hypochlorite solution would not only be costly, but would require frequent deliveries with tanker trucks. For this reason, the City may want to consider other options for reducing odor within the collection and conveyance system, such as the use of calcium nitrate.

The long range plan should include the containment and treatment of odors at the process locations with high odors. On September 19, 2000, operating staff were polled, and the unit processes were ranked from high odor potential to a lesser odor potential as follows:

| Process | Odor Ranking (3.0 High, 1.0 Low) |
|---|---|
| Grit Removal System | 3.0 |
| Influent Pump Station | 2.6 |
| Primary Thickener | 2.2 |
| DAF Thickener | 2.0 |
| WAS Storage (Aeration Basin No. 4) | 1.9 |

| Process | Odor Ranking (3.0 High, 1.0 Low) |
|---|---|
| Solids Handling Building | 1.8 |
| Aeration Basins | 1.3 |
| Biosolids Holding Tank | 1.2 |
| Primary Clarifier | 1.1 |
| Secondary Clarifier | 1.0 |

This is representative of the odor potential experienced at many treatment facilities, with the highest potential at the headworks, followed by solids handling processes, with other processes contributing to a much less extent.

## FACILITIES

### Operations Building

The existing operations building will not be adequate for the expanding facilities. Additional storage, expanded laboratory facilities, a records storage and archive room, and additional office space will be necessary as the City grows.

### Shop/Garage

The existing shop will not allow both the collection system staff and WWTP staff to function efficiently as the City grows. Additional garage space and storage will be required as the City expands.

## STAFFING

The existing WWTP staff will not be able to function efficiently as flows and workloads increase over time. The EPA has provided guidance for estimating staffing for a typical WWTP in the March 1973 publication of 'Estimating Staffing for Municipal Wastewater Treatment Facilities.' This estimation is general in nature and is affected by decisions such as the amount of on-site laboratory analysis performed, equipment maintenance, and effluent limits. A detailed breakdown of the calculation is provided in Appendix N.

Based on this estimation, the City of Mount Vernon Wastewater Treatment Plant will need 14 employees by 2010. The following summarizes the time line for staff addition:

| Year | Total Number of Staff | Comments |
|---|---|---|
| 2000 | 10 | Current |
| 2003 | 11 | Add Instrumentation/Electrical Staff |
| 2004 | 12 | Add Maintenance Staff |
| 2007 | 13 | Add Maintenance Staff |
| 2010 | 14 | Add Maintenance & Operations Staff |

## SUMMARY OF ANALYSES

The additional WWTP capacity required to meet 2010 and 2020 flows and loads are summarized in Table 8-4 and Table 8-5, respectively.

Table 8-4

| Summary of Requirements to Meet 2010 Flows and Loads | | | |
|---|---|---|---|
| **Unit Process** | **Existing Capacity** | **BOD removal** | **Nitrification** |
| Influent Pump Station (Firm Capacity)[1] | 10.8 mgd | 24.0 mgd[1] | 24.0 mgd[1] |
| West Mount Vernon Pump Station (Firm Capacity) | 1.2 mgd | 1.8 mgd | 1.8 mgd |
| Headworks - Fine Screens and Grit Removal (Total Capacity Required) | None | 25.8 mgd[2] | 25.8 mgd[2] |
| Primary Clarifiers (Total Required Surface Area) | 5,000 sf | 8,300 sf[2] | 8,300 sf[2] |
| Aeration Basins (Total Volume Required)[3] | 1.5 MG | 1.0 MG | 2.2 MG |
| Blowers (Firm capacity not provided for peak hour loads) | 12,300 scfm | 5,600 scfm | 10,300 scfm |
| Secondary Clarifiers (Total Required Surface Area)[5] | 5,675 sf | 16,500 sf | 16,600 sf |
| Disinfection (Total Capacity Required)[6] | Chlorine | 25.8 mgd[2] | 25.8 mgd[2] |
| Effluent Pump Station (Firm Capacity Required) | 12.0 mgd | 25.8 mgd[2] | 25.8 mgd[2] |
| Outfall (Total Capacity Required) | 12.0 mgd | 25.8 mgd[2] | 25.8 mgd[2] |
| Gravity Thickener (Total Required Surface Area)[7] | 380 sf | 150 sf | 150 sf |
| DAF Thickener (Total Required Surface Area)[8] | 1,250 sf | 1,500 sf | 1,800 sf |
| Anaerobic Digester (Total Required Volume)[9] | 103 kcf | 78 kcf | 82 kcf |

1. Hydraulic capacity increased to 24 mgd to provide additional CSO treatment capacity for Phase 2 CSO improvements.
2. Hydraulic capacity increased to 25.8 mgd to provide additional CSO treatment capacity for Phase 2 CSO improvements.
3. Existing aeration basin volume includes Aeration Basin No. 4, currently designated as an aerobic digester.
4. With coarse bubble diffusers replaced with fine bubble diffusers.
5. Existing secondary clarifiers include two 85-foot-diameter units, one of which is a peripheral feed unit with an 11-foot sidewater depth. It is anticipated that the 11-foot sidewater depth unit would be taken out of service.
6. Chlorine disinfection is to be replaced by UV disinfection.
7. Gravity thickener is designated for primary sludge thickening.
8. DAF thickener is designated for WAS thickening.
9. Due to the grit buildup and a scum layer in the digester, this is based on only 70% of the 103 kcf is available capacity (72.1 kcf).

Table 8-5

| Summary of Requirements to Meet 2020 Flows and Loads | | | |
|---|---|---|---|
| **Unit Process** | **Existing Capacity** | **BOD removal** | **Nitrification** |
| Influent Pump Station (Firm Capacity)[1] | 10.8 mgd | 24.0 mgd[1] | 24.0 mgd[1] |
| West Mount Vernon Pump Station (Firm Capacity) | 1.2 mgd | 1.8 mgd | 1.8 mgd |
| Headworks - Fine Screens and Grit Removal (Total Capacity Required) | None | 25.8 mgd[2] | 25.8 mgd[2] |
| Primary Clarifiers (Total Required Surface Area) | 5,000 sf | 10,100 sf[2] | 10,100 sf[2] |
| Aeration Basins (Total Volume Required)[3] | 1.5 MG | 1.2 MG | 2.7 MG |
| Blowers[4] | 12,300 scfm | 6,800 scfm | 12,500 scfm |
| Secondary Clarifiers (Total Required Surface Area)[5] | 5,675 sf | 21,000 sf | 21,000 sf |
| Disinfection (Total Capacity Required)[6] | Chlorine | 25.8 mgd[2] | 25.8 mgd[2] |
| Effluent Pump Station (Firm Capacity Required) | 12.0 mgd | 25.8 mgd[2] | 25.8 mgd[2] |
| Outfall (Total Capacity Required) | 12.0 mgd | 25.8 mgd[2] | 25.8 mgd[2] |
| Gravity Thickener (Total Required Surface Area)[7] | 380 sf | 200 sf | 200 sf |
| DAF Thickener (Total Required Surface Area)[8] | 1,250 sf | 2,000 sf | 1,750 sf |
| Anaerobic Digester (Total Required Volume)[9] | 103 kcf | 102 kcf | 99 kcf |

1. Hydraulic capacity increased to 24 mgd to provide additional CSO treatment capacity for Phase 2 CSO improvements.
2. Hydraulic capacity increased to 25.8 mgd to provide additional CSO treatment capacity for Phase 2 CSO improvements.
3. Existing aeration basin volume includes Aeration Basin No. 4, currently designated as an aerobic digester.
4. Coarse bubble diffusers replaced with fine bubble diffusers, firm capacity not provided for peak hour loads.
5. Existing secondary clarifiers include two 85-foot-diameter units, one of which is a peripheral feed unit with an 11-foot sidewater depth. It is anticipated that the 11-foot sidewater depth unit would be taken out of service.
6. Chlorine disinfection is to be replaced by UV disinfection.
7. Gravity thickener is designated for primary biosolids thickening.
8. DAF thickener is designated for WAS thickening.
9. Due to the grit buildup and a scum layer in the digester, this is based on only 70% of the 103 kcf is available capacity (72.1 kcf).

# 9. WASTEWATER TREATMENT PLANT ALTERNATIVES

Alternatives for unit processes identified deficient in Chapter 8 were developed based on future flows and loads, for years 2010 and 2020. Alternatives developed also were based on assuming that nitrification will eventually be required, as determined in Chapter 8. The following chapter makes recommendation for the preferred alternatives to meet future flows and loads.

## HYDRAULICS

The existing hydraulics of the wastewater treatment plant were presented in Figure 7-1. As noted in Chapter 7, the existing oxidation tower and oxidation tower pump station were functionally replaced by the activated sludge process. An evaluation of alternative hydraulic profiles through the WWTP was performed. The relative costs for each unit process affected was assessed to determine which hydraulic profile was the most cost effective.

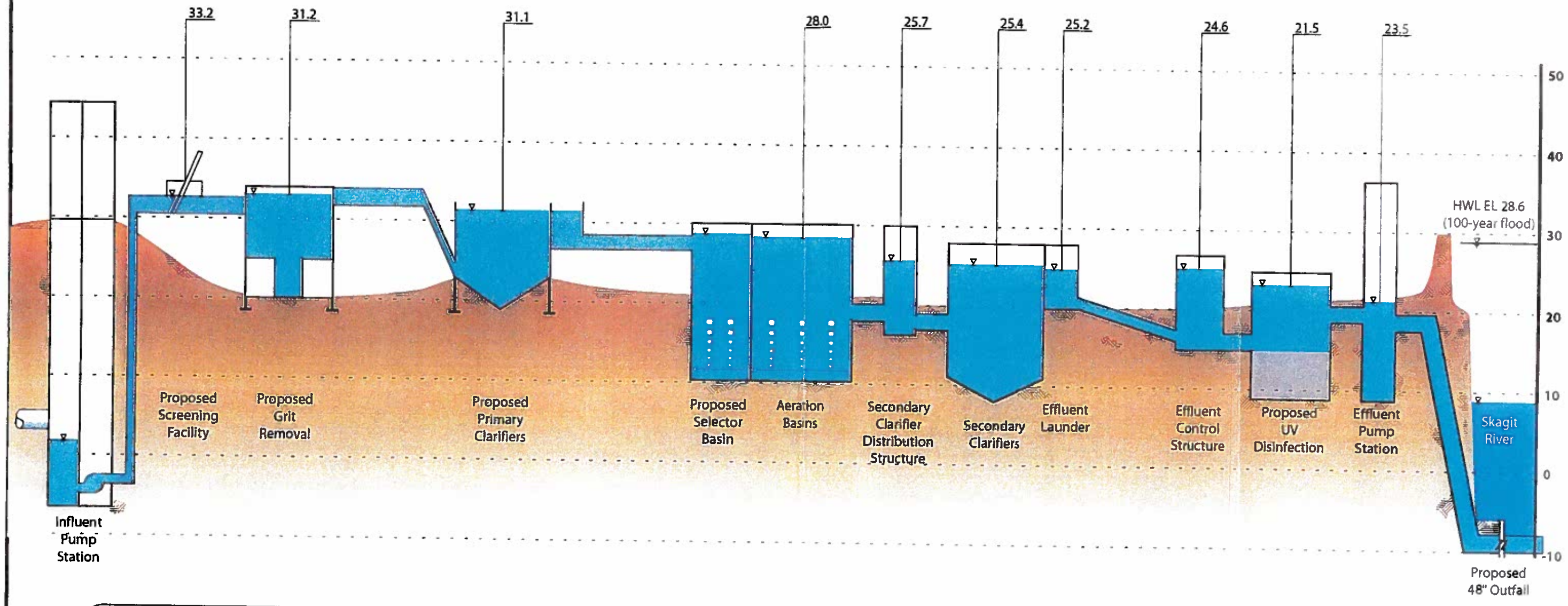
### Alternative A - Existing WWTP Hydraulics

Alternative A maintains the existing WWTP hydraulics. With the existing hydraulics, wastewater is pumped from the influent pump station to the comminutor. Wastewater gravity flows through the primary clarifier to the activated sludge pump station. At this lift station, wastewater is raised to approximately 30.8± feet, where it flows by gravity to the effluent pump station. Effluent flows exit the pump station by gravity, unless the river level is elevated, requiring effluent pumping.

Plant capacity can be maintained with the existing hydraulic profile. Replacement of the comminutor with a modern headworks, fine screening and grit removal, can be accomplished within the existing hydraulics. Expansion of the primary clarifiers (addition of 5,600 sf) also can be accomplished within the existing hydraulics. With this hydraulic configuration, the cost estimate for a new headworks and primary clarifiers would be $3.5 and $1.1 million, respectively. This alternative would also require the construction of a new RAS pump station, allowing the existing activated sludge pump station to be utilized for forward flow only. The cost estimate for a new RAS pump station ranges from $600,000 to $800,000. The total cost estimate for this alternative is $5.3 million.

### Alternative B - Eliminate Intermediate Pumping

Alternative B eliminates the intermediate pump station (existing activated sludge pump station) for pumping of primary effluent to the aeration basins. The hydraulic grade of the primary clarifiers is raised and the influent pumps are sized for these conditions. The required improvements could be accomplished with this new hydraulic grade, as presented in Figure 9-1. The estimated cost of a new headworks and primary clarifiers is $3.4 and $1.8 million, respectively. This alternative allows the existing activated sludge pump station to be utilized for RAS pumping only. The total cost estimate for this alternative is $5.2 million.




**Legend**

29.00 — Water Surface Elevation Peak Hour Conditions:
All but secondary process= 24 mgd
Secondary process = 18.9 mgd

HDR Engineering, Inc

| | |
|---|---|
| Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE | Date: FEBRUARY 2003 |
| Sheet Title: PROPOSED HYDRAULIC PROFILE | Figure No: 9-1 |

## INFLUENT PUMP STATION

### INTRODUCTION

The existing Influent Pump Station has a firm pumping capacity of 10.8 mgd. There are a number of operating problems associated with this facility as follow:

- During high flow CSO situations, the influent gate is modulated to limit the flow to the pump station to prevent exceeding the capacity of the station. Continuous operation of the modulating gate system depends on interaction of a number of components (flow meter, modulating gate operator and controller) and there is a risk that this flow limit will not always be maintained. There have been occasions when the wetwell has become surcharged requiring cleaning of the grating and walls of the wetwell after the event.

- During high flow conditions, the center two pumps are reported to become "air locked". This may be due to the configuration of the inlet to the wetwell. The flow currently discharges directly between the inlets to Pump Nos. 2 and 3. This "waterfall" between the pump inlets causes significant turbulence and is not a desirable inlet condition.

Upgrade of the Influent Pump Station must address the two items above. The 42-inch diameter influent interceptor to the station has a capacity of 24 mgd. The required peak hour capacity for the year 2010 is 14.9 mgd and for the year 2020 is 18.3 mgd. It is proposed to upgrade the station to a firm pumping capacity of 24 mgd. This additional hydraulic capacity will provide hydraulic capacity to further reduce the number of CSO overflow events (Phase 2 CSO Improvements). Two alternatives were developed for the upgrade of the station. Alternative A would maintain the existing wetwell-drywell configuration and Alternative B would convert the existing drywell to a wetwell and the pumps would be replaced with submersible pumps.

### Alternative A - Retrofit Existing Pump Station with new Pumps and Motors

The primary concern with retrofitting the existing station with larger pumping equipment would be to insure that the current wetwell hydraulic problems do not continue. Based on a preliminary review it appears that by raising the operating level in the wetwell and diverting the inflow away from the pump inlets, the problem can be eliminated. Prior to proceeding with this alternative, it is suggested that a physical model be constructed and the before and after conditions simulated to insure the problems are corrected with the proposed modifications. The estimated costs for a physical model are $30,000 to $50,000.

Preliminary sizing of the pumping units was completed and four 100 hp units would be required to provide a firm pumping capacity of 24 mgd. The structure above the drywell presently includes the electrical room and the standby generator room. The present standby generator unit is a 300kW unit which provides emergency power for all essential loads at the plant. Any upgrade to the plant will increase the required standby power. In this case it is suggested to maintain the existing generator unit for the Influent Pump Station, and "offload" other existing essential loads and additional new loads to a new

engine-generator unit. The existing 300 kW unit will have adequate capacity for the 100 hp pumps with variable frequency drives. A preliminary plan for this alternative is shown on Figure 9-2, and a section on Figure 9-3. Capital costs for Alternative A were developed and are shown on Table 9-1.

**Table 9-1**

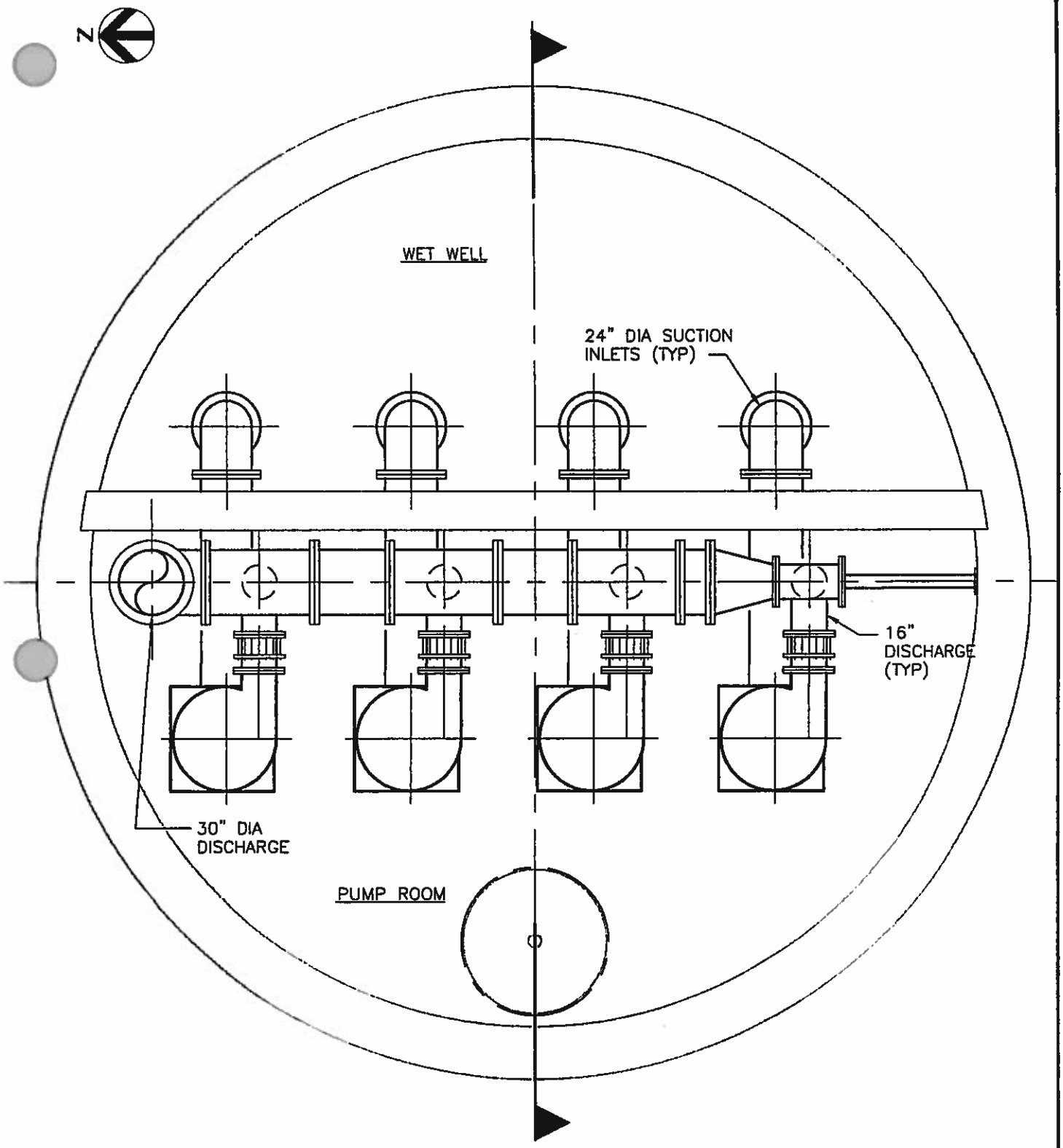
| Influent Pump Station: Alternative A Cost Estimate (Upgrading Existing Wetwell/Drywell Pump Station) | | | | |
|---|---|---|---|---|
| **Item** | **Quantity** | **Unit** | **Unit Cost** | **Cost** |
| Bypass Pumping | 1 | LS | $50,000 | $50,000 |
| Replace Existing Pumps | 4 | EA | $65,000 | $260,000 |
| Replace Existing Piping | 1 | LS | $80,000 | $80,000 |
| Forcemain | 1 | LS | $200,000 | $200,000 |
| Modify Existing Wetwell | 1 | LS | $50,000 | $50,000 |
| Replace Existing VFDs | 4 | EA | $40,000 | $160,000 |
| Additional Barscreen | 1 | LS | $200,000 | $200,000 |
| Electrical | 1 | LS | $30,000 | $30,000 |
| | | | | |
| Subtotal | | | | $1,030,000 |
| | | | | |
| Contingency (20%) | | | | $206,000 |
| Indirect Project Costs (30%) | | | | $371,000 |
| | | | | |
| Total | | | | $1,606,000 |

### Alternative B - Remodel Existing Pump Station for Submersible Pumps

Alternative B would convert the existing drywell to a wetwell and install submersible pumps. This would require significant structural changes. The existing Electrical Room and Standby Generator Room would be demolished. All of the piping and equipment would be removed from the drywell. A new structure would be provided for the electrical controls and relocation of the standby generator. A valve vault would be constructed adjacent to the new wetwell as shown on Figure 9-4. A section view of this concept is shown on Figure 9-5. Capital costs for Alternative B were developed and are shown in Table 9-2.

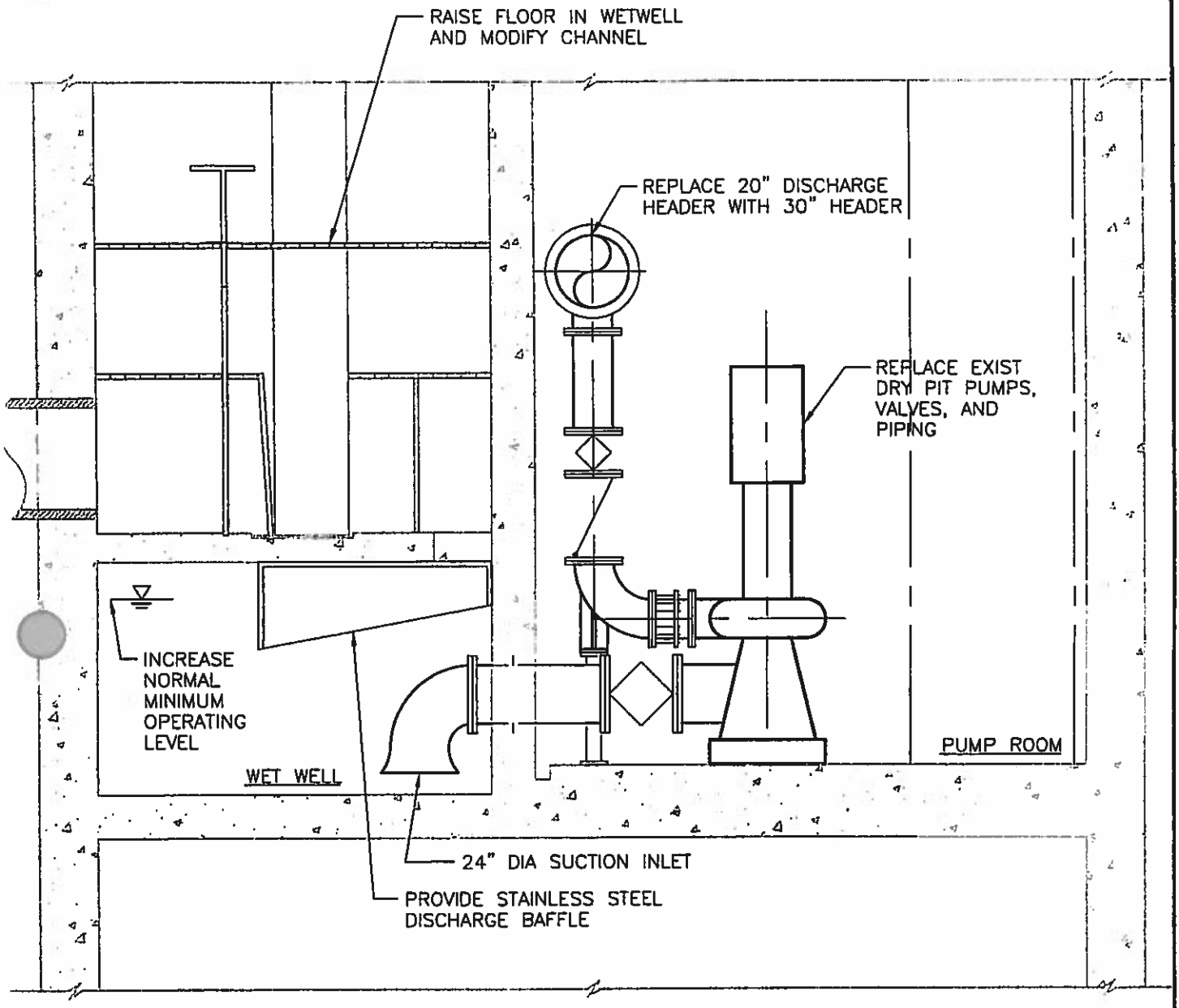
Table 9-2

| Influent Pump Station: Alternative B Cost Estimate (Convert to Submersible Pump Station) | | | | |
|---|---|---|---|---|
| Item | Quantity | Unit | Unit Cost | Cost |
| Remove existing superstructure | 1 | LS | $30,000. | $30,000 |
| Remove existing equipment | 1 | LS | $20,000. | $20,000 |
| Bypass Pumping | 1 | LS | $50,000. | $50,000 |
| Additional Barscreen | 1 | LS | $200,000. | $200,000 |
| Modify Drywell | 1 | LS | $80,000. | $80,000 |
| Valve Vault and Piping | 1 | LS | $120,000. | $120,000 |
| Forcemain | 1 | LS | $200,000 | $200,000 |
| Electrical Control Building | 800 | SF | $150. | $120,000 |
| Submersible Pumps | 4 | EA | $70,000. | $280,000 |
| Modify Existing Wetwell | 1 | LS | $30,000. | $30,000 |
| VFDs | 4 | EA | $40,000. | $160,000 |
| Electrical | 1 | LS | $50,000. | $50,000 |
| | | | | |
| Subtotal | | | | $1,340,000 |
| | | | | |
| Contingency (20%) | | | | $268,000 |
| Indirect Project Costs (30%) | | | | $482,000 |
| | | | | |
| Total | | | | $2,090,000 |



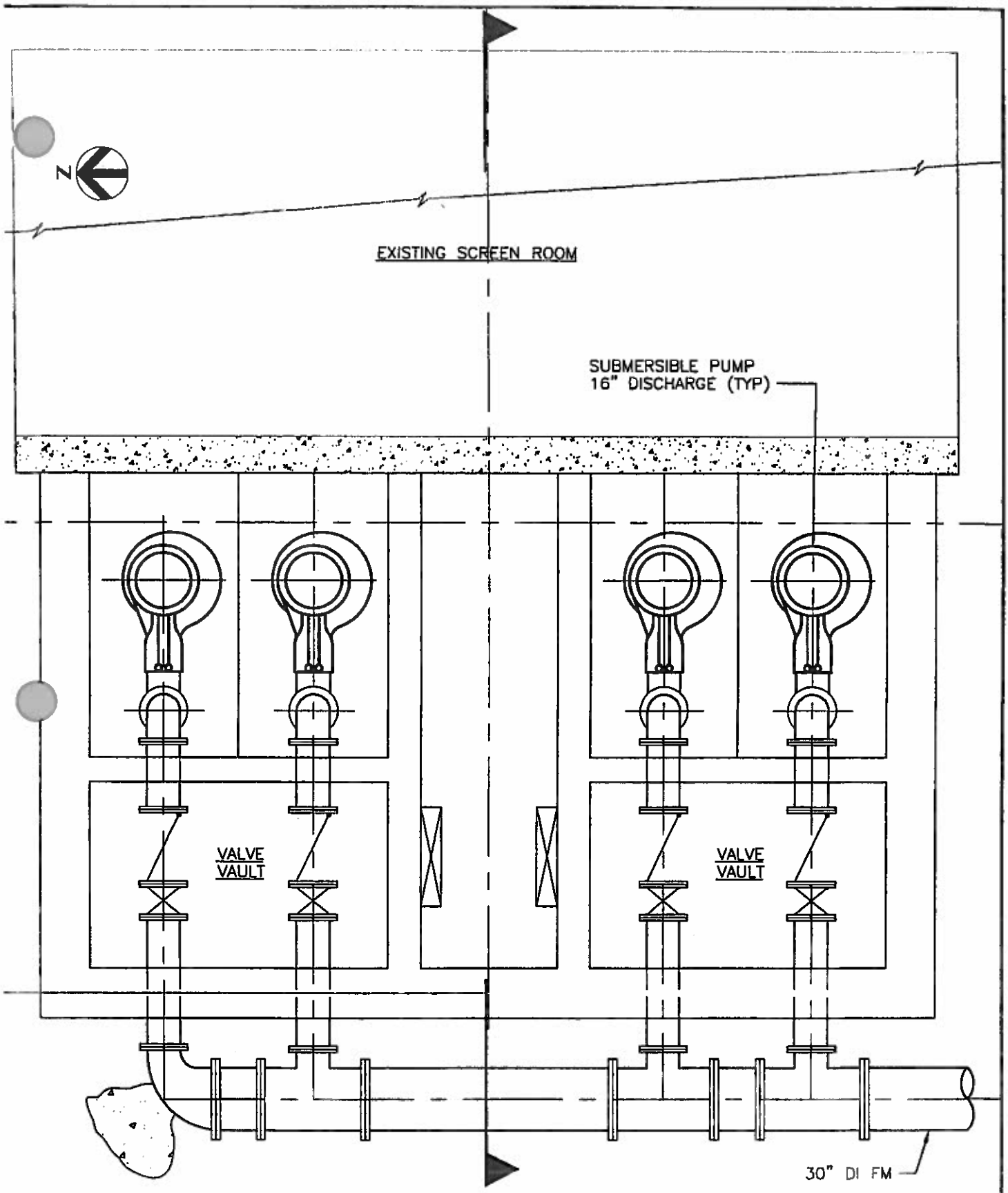

HDR Engineering, Inc.

| | |
|---|---|
| Project Title<br>MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE | Date<br>FEBRUARY 2003 |
| Sheet Title<br>INFLUENT PUMP STATION UPGRADE ALTERNATIVE A–PLAN | Figure No.<br>9–2 |



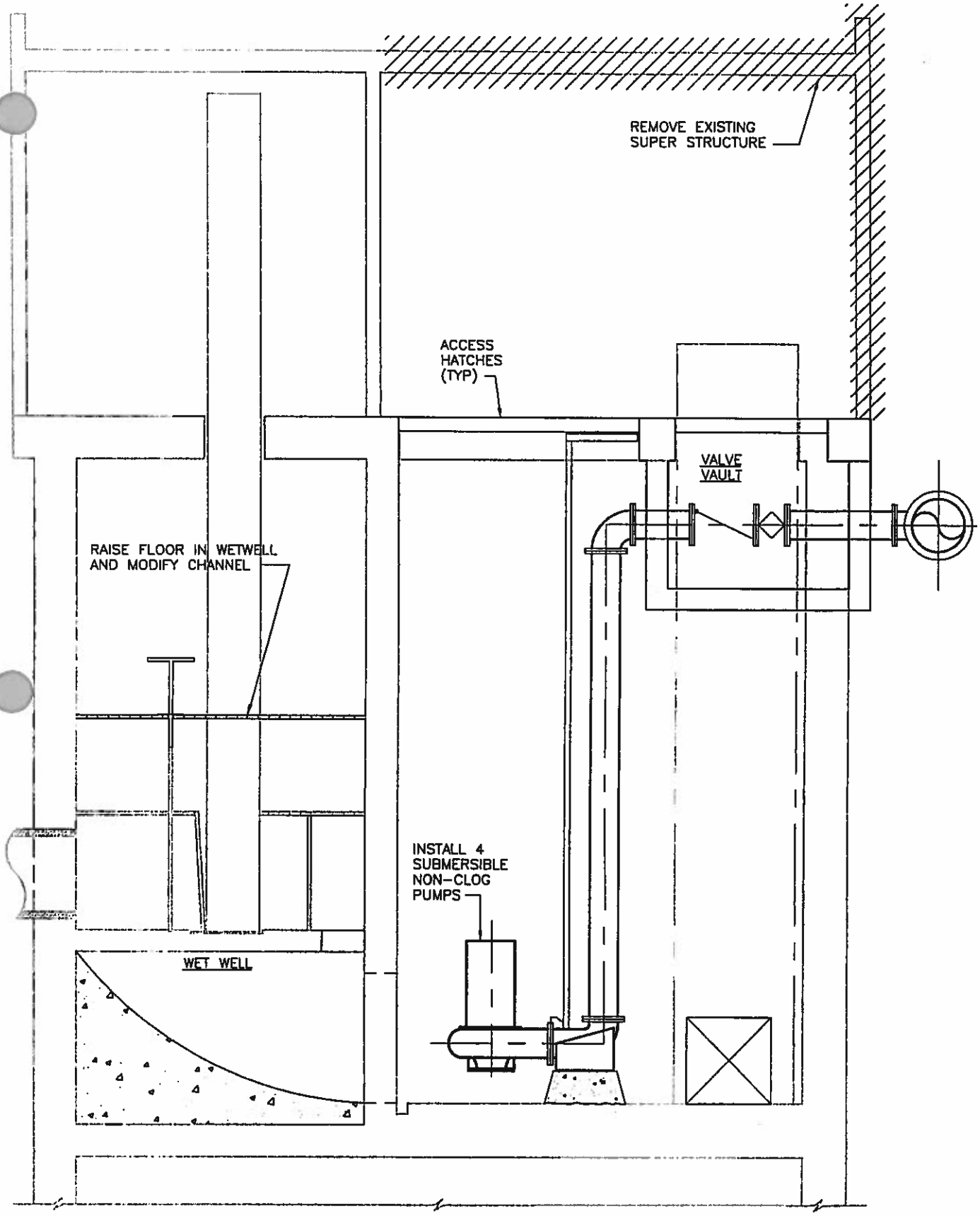

HDR Engineering, Inc.

| | |
|---|---|
| Project Title<br>MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE | Date<br>FEBRUARY 2003 |
| Sheet Title<br>INFLUENT PUMP STATION UPGRADE ALTERNATIVE A–TYPICAL SECTION | Figure No.<br>9–3 |




HDR Engineering, Inc.

| | |
|---|---|
| Project Title<br>MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE | Date<br>FEBRUARY 2003 |
| Sheet Title<br>INFLUENT PUMP STATION UPGRADE ALTERNATIVE B–PLAN | Figure No.<br>9–4 |




HDR Engineering, Inc.

Project Title
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date
FEBRUARY 2003

Sheet Title
INFLUENT PUMP STATION UPGRADE ALTERNATIVE B–TYPICAL SECTION

Figure No.
9–5

## HEADWORKS

More efficient methods of solids and grit removal (compared to the current practice of de-gritting primary sludge) can be accomplished with modern equipment, as described below. Better screening and grit removal will reduce the wear on downstream process equipment.

### Screening

Coarse screening provided upstream of the influent pumps removes larger debris from the liquid waste-stream, but does not remove any debris from the wastewater pumped by the West Mount Vernon Pump Station. To remove plastics, rags, and small rocks from the influent wastewater (from both the Influent Pump Station and the West Mount Vernon Pump Station), fine screens would be required in a Headworks Facility.

Fine screens would have 3/8-inch spacing and be mechanically cleaned. They can be expected to remove approximately 9 $ft^3$/MG wastewater, or approximately three times the volume of screenings removed by the existing 1-inch coarse screens. The fine screens would be the first unit process treating the entire forward flow of the WWTP. Screenings washing equipment will be provided to remove organic material from the screenings and a screening compactor to reduce the volume to be disposed.

### Grit Removal

Alternatives for grit removal from the liquid waste-stream, rather than the primary sludge, include:

*Aerated Grit Chambers.* Aerated grit chambers trap grit through an air-induced rotation of the wastewater at a velocity of approximately 1 fps. Detention time is typically three to five minutes, with one to five standard cubic feet per minute (scfm) of air per linear foot of basin.

*Vortex Grit Chambers.* Vortex grit chambers are gravity units that swirl the wastewater causing inorganic matter to settle to the tank hopper section of the unit. The vortex can be created through natural hydraulics or induced by slowly rotating paddles. Grit is removed by pumping it from the hopper section of the unit.

*Hydrocyclone Degritters.* Hydrocyclone degritters utilize centrifugal forces in a cone shaped unit to separate the grit and wastewater. Wastewater enters and exits in the upper portion of the unit, and a grit containing slurry exits through a small opening near the bottom of the unit. The cyclone process includes a pump as an integral part of the unit, for it depends on a steady liquid stream supply.

Capital and operating costs for each alternative were reviewed. The costs, summarized in Table 9-3, were assessed on a low, moderate, high scale. The flexibility of the grit removal system to accept a wide range of flows was also assessed on the same scale.

Table 9-3

| Evaluation of Grit Removal Alternatives | | | | |
|---|---|---|---|---|
| Alternative | Description | Capital Cost | Annual O & M Cost | Operating Flow Range[1] |
| 1 | Aerated Grit Chamber | $1,000,000 | $37,000 | Low |
| 2 | Vortex Grit Chamber | $700,000 | $25,000 | High |
| 3 | Hydrocyclone Degritter | $5,000,000 | $90,000 | Moderate |

1. The operating flow range of the grit removal system to perform acceptably over a wide range of flows.

### Disposal

The existing method of final disposal, to convey grit and screenings to the landfill, is still a viable alternative. A building should be placed around the screenings and grit storage site to contain odor.

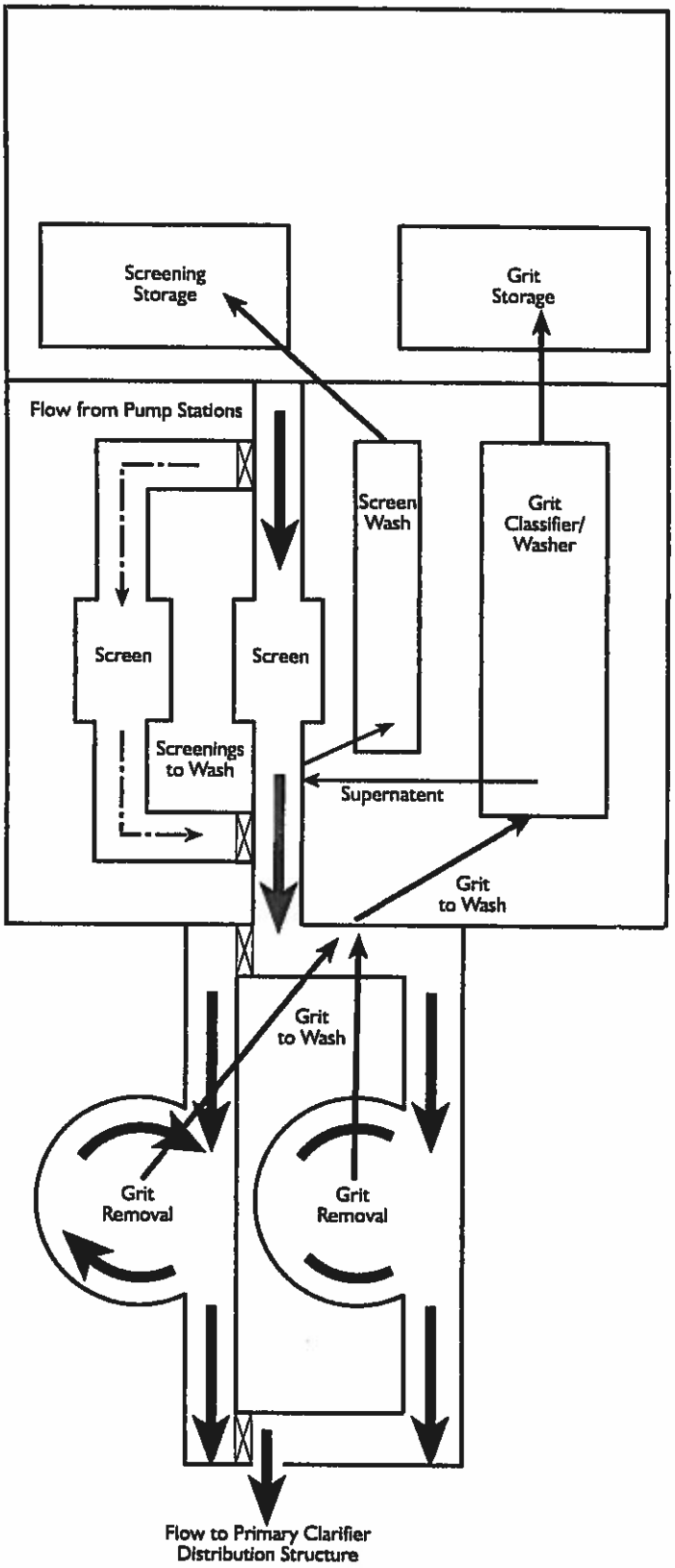
### Primary Sludge and Scum Pumping

The installation of two new primary clarifiers will require additional sludge and scum pumping facilities. These should be located within a close proximity to the primary clarifiers and would be installed in the lower floor of the new headworks facility.

### Cost Estimate

A typical headworks configuration is shown on Figure 9-6. It has the potential to be placed in one of two locations: Near the Influent Pump Station, or near the Primary Clarifiers. Since the area near the Influent Pump Station is designated for solids treatment, the logical location for a headworks facility is near the primary clarifiers.

The estimated capital cost for a headworks facility (including fine screens, grit removal, primary sludge and scum pumping, and screening and grit storage until final disposal) would be $2.8 million.




HDR Engineering, Inc.

| | |
|---|---|
| Project Title<br>MOUNT VERNON COMPREHENSIVE SEWER | Date<br>FEBRUARY 2003 |
| Sheet Title<br>PROPOSED HEADWORKS FACILITY | Figure No.<br>9-6 |

## PRIMARY CLARIFIERS

Additional primary clarifier capacity should be provided for future flows and to provide redundancy. The hydraulic analysis determined that raising the WSEL of the treatment processes (allowing gravity forward flow) was the most desirable hydraulic profile.

### Alternative A - Modify Existing Primary and Add New Primary Clarifier

Alternative A includes modifications to the existing primary clarifier (to raise the water surface elevation) and addition of a second primary clarifier to meet future needs and provide redundancy. Modifications to the existing 5,000 sf primary clarifier would include:

- Raising the sidewalls of the clarifier tank approximately 4.5 feet;
- Raising the effluent weirs; and
- Replacing the clarifier mechanism.

The new primary clarifier would have a larger footprint than the existing primary:

| | | |
|---|---|---|
| • Diameter: | 90-foot | |
| • Sidewater Depth: | 12 feet | |
| • Design flows: | ADMM: | 5.5 mgd |
| | Peak Hour: | 13.8 mgd |

Both clarifiers would have WSEL of approximately 31.2± feet. A primary clarifier distribution structure would split flows between the existing and new clarifiers.

Combined sewer flows would be treated in a separate process. An 'internal shunt' would be utilized to process a portion of the combined sewer flows. Flows would be split, with 18.3 mgd (peak hour sanitary flows) to the aeration basins and 7.5 mgd (combined sewer flows) to the disinfection system. This will provide for the Phase 2 CSO Improvements. This flow split would be performed in the aeration basin distribution structure. Effluent blending would take place prior to the disinfection process.

### Alternative B - Two New Primary Clarifiers

Alternative B consists of adding two new primary clarifiers to treat sanitary flows and utilizing the existing primary clarifier for CSO flows. Two new primary clarifiers would have the following attributes:

- Diameter: 75-foot
- Sidewater Depth: 12 feet
- Design flows: ADMM: 4.9 mgd

  Peak Hour: 9.2 mgd

Both clarifiers would have WSEL of approximately 31.2± feet. A primary distribution structure would be required, splitting flows between the new clarifiers and the existing clarifier (for CSO treatment).

The existing primary clarifier would be utilized, without modification, for treatment of CSO flows, via the 'internal shunt' mechanism. Utilizing the existing primary for this purpose would yield an HRT of 1.2 hours at 7.5 mgd. Flows would receive primary treatment, and flow by gravity to the disinfection system for effluent blending and disinfection.

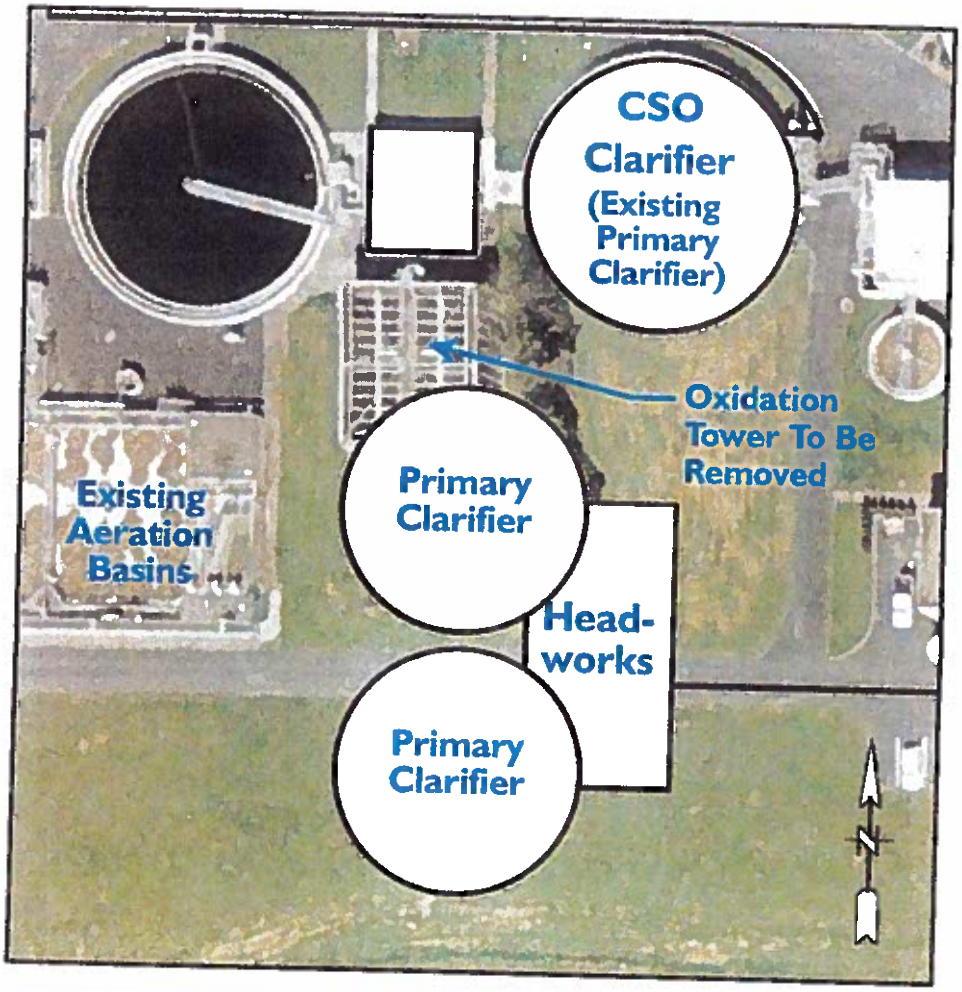
### Cost Estimate

Capital and operating costs for each alternative were developed. The capital costs, summarized in Table 9-4, include a 20% contingency and 30% for indirect costs, i.e. sales tax, engineering, administration, and legal.

**Table 9-4**

| Capital Costs ($1,000) for 25.8 mgd Primary Clarifier Alternatives | | |
|---|---|---|
| **Alternative** | **Description** | **Capital Cost** |
| A | Existing and New Clarifier | $1,563 |
| B | Two New Clarifiers | $1,794 |

The primary clarifiers could be located in a variety of locations, ranging from adjacent to the existing primary clarifier to locations south of the aeration basins and shop/garage. The most logical location for new primary clarifiers is in the location near the existing oxidation tower (adjacent to the existing primary clarifier). A conceptual plan of two additional primary clarifiers and headworks facility is shown on Figure 9-7.




HDR Engineering, Inc.

| | |
|---|---|
| Project Title: MOUNT VERNON COMPREHENSIVE SEWER | Date: FEBRUARY 2003 |
| Sheet Title: PROPOSED PRIMARY CLARIFIERS | Figure No: 9-7 |

## ACTIVATED SLUDGE PROCESS

### Activated Sludge Pump Station/RAS Pump Station

The existing activated sludge pump station is equipped with three screw pumps, each with a capacity of 8 mgd. With the proposed modification to the hydraulic profile the station is no longer necessary for forward flows.

**Alternative A - Abandon Activated Sludge Pump Station**
For Alternative A, the Activated Sludge Pump Station would be abandoned. This alternative would require a new RAS pump station to be built either at this location or a different site for an estimated cost from $600,000 to $800,000.

**Alternative B - Convert Activated Sludge Pump Station to RAS Pump Station**
Alternative B would recommend converting the activated sludge pump station to a dedicated RAS pump station. The existing facilities are suited for this conversion because the pump station pumps from an elevation low enough to collect RAS flows and to an elevation where RAS flows could be fed into a selector basin or aeration basin distribution structure.

Typical sizing criteria is to provide 100 percent of the ADMM flow capacity for RAS pumping. The existing station will provide adequate capacity through 2020. For year 2020 flows, the recommended pumping capacity is 9.9 mgd (2020 ADMM). The recommended pumping capacity is far less than the available capacity, so the pump station could be used without modification through 2020.

### Selector Basin

When there is an abundance of filamentous organisms in the activated sludge process, the settling characteristics of the biomass is inhibited. The production of a high SVI filamentous bulking sludge results in high effluent solids concentrations and the potential for a permit violation.

There are two approaches to the control of filaments in the activated sludge process. One approach is to chlorinate the RAS at chlorine concentrations of 5 to 10 mg/L to minimize the presence of filamentous sludge. The second approach is to provide a selector basin upstream of the aeration basins to limit the filamentous bacteria population via the biological process.

**Alternative A - Chlorinate RAS for Filament Control**
Alternative A would control filaments by chlorinating the RAS. This is the current method of filament control and would require no modification. The disadvantages to chlorinating the RAS are as follows:

- Disinfection By-Products (DBP) are formed in the wastewater; and

- Chlorine (which has numerous safety issues) is required.

**Alternative B - Construct Selector Basin for Filament Control**

Alternative B would provide a selector basin to control filament growth. There are three operating modes for selectors. Aerobic, anoxic, and anaerobic. Depending on the operating mode, the hydraulic retention time recommended is from 10 to 60 minutes. This detention time, combined with the influent BOD concentration, promotes the growth of floc forming bacteria while limiting the growth of filamentous bacteria. The anoxic selector can only be used in a plant that includes nitrification in the activated sludge process, since it requires the nitrates produced in the nitrifying process.

Preliminary sizing was completed for the year 2020 flow conditions. The selector basins could be constructed in two phases. The initial phase would consist of multiple cells with a total volume of 0.3 mg operating in aerobic mode and would accommodate the plant in the 'non-nitrifying' mode. When provisions were made for nitrification, an additional 0.3 mg cell would be added to permit operation in the anoxic mode. These selector basins would be at a water surface of approximately 30 ± ft to maximize flow distribution options. The estimated cost of a selector is $600,000.

### Chemical Feed System

The nitrification process will typically reduce alkalinity of the mixed liquor resulting in a reduction of the pH. Plant staff performed a trial operation of the activated sludge process in the nitrification mode and experienced a reduction in pH which approached the NPDES permit limits and the nitrification test was terminated.

To operate in the nitrification mode, a chemical feed system must be provided to provide for pH adjustment. In addition, the proper pH limits must be maintained in the aeration basins to maintain the nitrification process. A chemical feed system should be provided to supply caustic soda. The primary discharge point would be at the inlet to the aeration basins. By controlling the pH at the inlet, permit limits should be able to be maintained in the effluent. In addition to the aeration basin feed point, the caustic soda could also be supplied upstream of the effluent disinfection process. This would provide additional assurance that the effluent pH limits are maintained.

The components for the pH control system would include a caustic soda storage tank with containment protection, two chemical feed pumps, and chemical feed piping to the aeration basin inlet channel and upstream of the existing chlorine contact tank. A budget cost of $50,000 has been included for this improvement.

### Aeration System

Electrical costs could be reduced by installing fine bubble diffusers. Overall efficiencies of the fine bubble systems typically exceed the efficiencies of the coarse bubble systems by a factor greater than two. Review was made of overall plant energy usage and energy usage

for the aeration system. Average total monthly energy consumption was approximately 250,000 kWhrs and of this, approximately 135,000 kWhrs were used for aeration. This is approximately 54% of the total energy consumption. Aeration energy costs typically range from 45% to 60% of the total plant energy usage, depending on the process and equipment, so this is in the normal range. By converting the diffusers to a fine bubble system, the present estimated annual savings would be approximately $41,000 per year. This is based on current average electrical cost of $0.05 per kWhr. As flows and loads increase and power costs increase, the annual savings would also increase. When the plant eventually provides nitrification, the aeration requirements will increase by a factor of two. The provision of fine bubble diffusers will minimize these future aeration costs. To maximize savings, the City may want to consider completing the installation of the fine bubble diffuser system on a 'fast track' schedule, prior to implementing other improvements.

With the current operating mode (no nitrification), the payback period could range from 5 to 10 years for this improvement, but there are grant programs available that can provide up to 50% funding for the installation of energy saving equipment. These are provided by the power utilities since implementation of energy conservation reduces future demand and the need to construct additional energy sources for the power utility. With a 50% grant, the payback would be in the range of 2 to 5 years, depending on the process (nitrification or not) and current energy costs.

A detailed evaluation was completd to evaluate the replacement of coarse bubble diffuser with fine bubble diffusers and this confirmed the energy savings due to the increased efficiency and confirmed that the existing centrifial blowers that the existing centrifical blowers could be maintained with the proposed aeration system.. A copy of this technical memorandum summarizing this evaluation is included as Appendix M.

### Aeration Basins

The aeration basins are currently operated in a BOD removal (no nitrification) mode with coarse bubble diffusers. Fine bubble diffusers offer better oxygen transfer to the wastewater, resulting in more efficient operation and lower operating costs. The activated sludge process can typically be operated in three modes:

- BOD removal
- Nitrification ($NH_3$ removal)
- Denitrification ($NO_3$ removal)

The choice of which mode to operate in, and plan for, is typically driven by permit requirements. Mount Vernon's future NPDES permits will be limited by the TMDL of the Skagit River and the toxicity of ammonia to biological organisms in the Skagit River. These limits will require the WWTP to nitrify to meet ammonia limits.

#### Alternative A - BOD Removal Only

Alternative A provides basin capacity for BOD removal. The existing coarse bubble diffusers would be replaced with fine bubble diffusers to improve efficiency. Fine bubble diffusers have a higher oxygen transfer efficiency than the current coarse bubble diffusers.

This transfer efficiency coupled with a low headloss through the membrane results in a lower power consumption. This alternative would require a total basin capacity of 1.0 mg by 2010 and 1.2 mg by 2020. Aeration Basin No. 4 (0.47 mg) would be utilized as an aeration basin rather than a WAS holding tank or an aerobic digester. The disadvantage of this alternative is that the effluent will not meet anticipated future ammonia limits.

**Alternative B - Nitrification**

Alternative B would provide basin capacity to nitrify the wastewater, reducing ammonia levels to below anticipated permit limits. To provide nitrification, approximately 2.2 mg of volume would be required for 2010 flows and 2.7 mg for 2020 flows. This would essentially require additional basin capacity to the south of the existing basins. Preliminary layouts developed for the aeration basins were developed based on the capacity analyses and are shown in Figure 9-8.

Aeration for all the basins would be fine bubble diffusers, as explained in alternative A.

**Alternative C - Denitrification**

Alternative C would provide for denitrification. Denitrification would reduce the nitrate levels in the effluent and should be implemented if nitrate is eventually regulated. At the current time, nitrate is not, and does not appear to be, a nutrient of concern. If the facility were to be sized for denitrification, additional basin volume would be provided to the west of the existing and future phase basins.

Aeration for the basins would be fine bubble, as explained in alternative A.

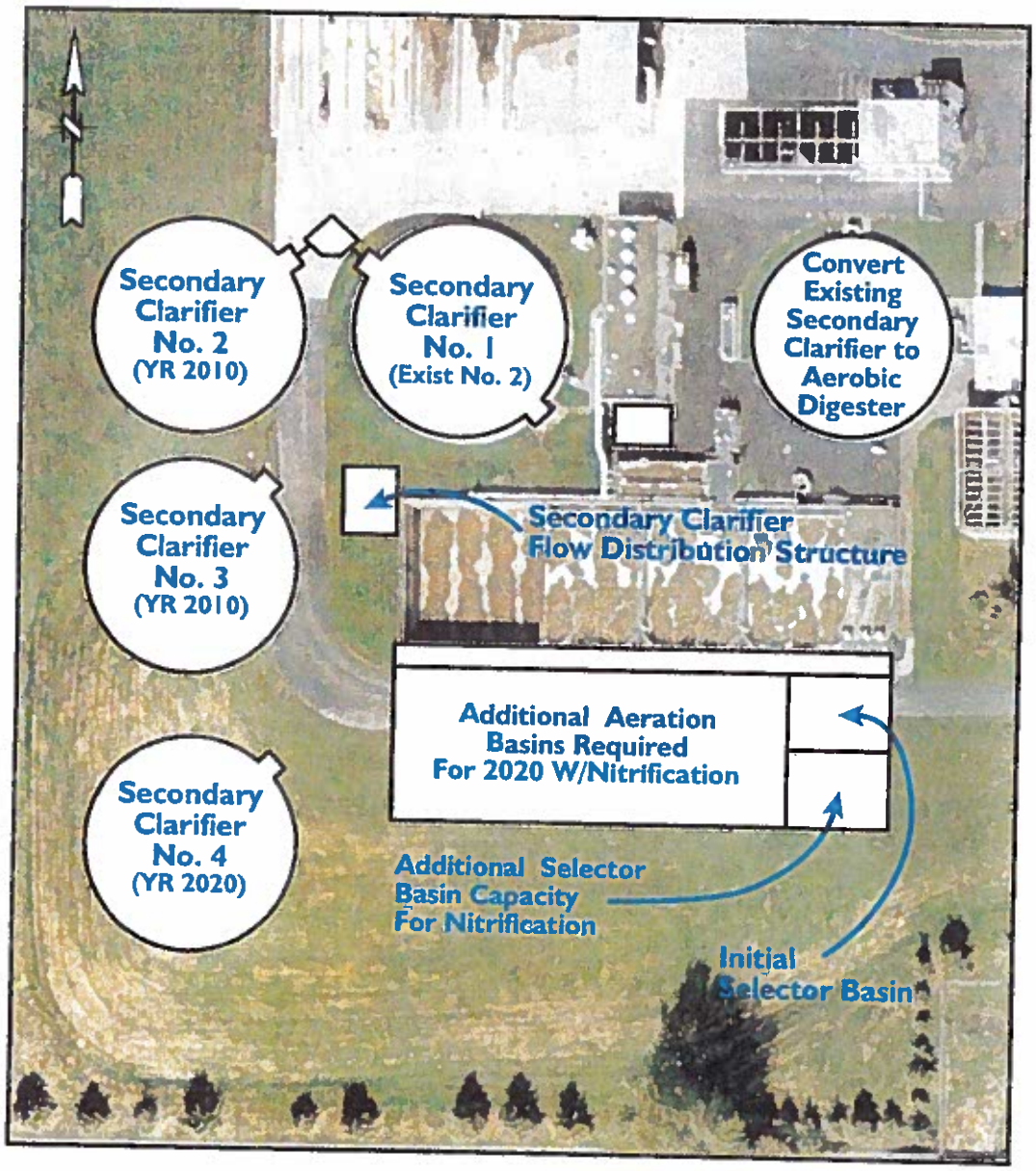
**Cost Estimates**

Total project costs were determined for each alternative as presented in Table 9-5.

**Table 9-5**

**Aeration Basin Improvements' Estimated Project Cost**

| Alternative | Condition | Cost |
|---|---|---|
| Alternative A - BOD Removal. Retrofit Existing Aeration Basins with Fine Bubble Diffusers | 2020 without nitrification | $300,000 |
| Alternative B - Additional 1.2 mg Aeration Basin Volume, and Retrofit Existing Aeration Basins with Fine Bubble Diffusers | 2020 with nitrification | $2,700,000 |
| Alternative C - Additional 1.2 mg Aeration Basin Volume for nitrification, 0.9 mg Aeration Basin Volume for denitrification, and Retrofit Existing Aeration Basins with Fine Bubble Diffusers | 2020 with denitrification | $4,600,000 |
| Chemical Feed System (pH control) | Required to operate in nitrification mode | $50,000 |




HDR Engineering, Inc.

| | |
|---|---|
| Project Title: MOUNT VERNON COMPREHENSIVE SEWER | Date: FEBRUARY 2003 |
| Sheet Title: PROPOSED AERATION BASINS AND SECONDARY CLARIFIERS | Figure No: 9-8 |

### Secondary Clarifiers

Since existing Secondary Clarifier No. 1 has a relatively shallow sidewater depth (11 ft.) and peripheral feed, this unit was assumed to be taken out of service. It could be used as an aerobic digester (biosolids storage), replacing the function that Aeration Basin No. 4 provided since that will be needed for aeration basin capacity.

Criteria for sizing of secondary clarifiers is typically dependant on both hydraulic loadings (peak hour and average day) and solids loadings. The City of Mount Vernon Sewer System is a combined sewer system which includes "in-line" storage provided by the Central CSO Regulator. This feature minimized overflows to the Skagit river, but also extends the duration of peak flows to the plant. Under this circumstance, the peak hour rating for the clarifier was reduced from 1,200 gpd/sf to 900 gpd/sf. Preliminary sizing was completed for secondary clarifiers based on this criteria. Two additional 85 ft. diameter units would be required for the year 2010 flows with an additional unit provided for the year 2020 flows. Cost for these units are summarized in Table 9-6.

**Table 9-6**

| Cost for Secondary Clarifiers | | |
|---|---|---|
| **Description** | **Flow Condition** | **Cost** |
| Two (2) @ 85-ft-diameter clarifiers and piping and distribution structure | 2010 | $2,500,00 |
| One (1) @ 85-ft-diameter clarifiers and piping | 2020 | $1,100,00 |

The clarifiers can be physically situated in a variety of locations at the WWTP, the suggested location is south or west of the proposed aeration basins. The amount of piping required can be reduced and flow distribution simplified by locating two secondary clarifiers to the north of the aeration basins, and two to the south of the basins. A proposed layout of this configuration is presented previously in Figure 9-8.

## DISINFECTION

UV disinfection systems were evaluated to determine the one best suited for the Mount Vernon WWTP. The expected headloss through UV systems is 4 inches to 2.0 feet. The maximum water surface elevation required downstream (at the effluent pump station) is 21.7± feet. The minimum water surface elevation upstream of the UV disinfection system (at the secondary distribution structure) is 25.0± feet. Thus, there is adequate head both upstream and downstream for any UV disinfection system.

### Alternative A - Horizontal, Low Pressure System

Alternative A included the review of a conventional horizontal, low pressure system. Due to the large footprint and associated number of bulbs, this was eliminated from further consideration.

### Alternative B - Low Pressure, High Intensity System

Alternative B was a horizontal, high intensity, low pressure UV disinfection system. These systems utilize dimensionally similar bulbs to the horizontal, low pressure systems but due to the 100 W bulb rather than the 32 W bulb have a smaller footprint. They have the potential for flow-paced power consumption. Units typically have a turn down ratio of 100 percent to 60 percent. They also have the potential for in-channel cleaning, limiting the number module removal times required for cleaning.

A horizontal, high intensity, low pressure system for Mount Vernon would include approximately 256 lamps and require a peak power requirement of 32 kw. This system can be supplied by multiple manufacturers.

The estimated required dimensions for each channel (requires two channels, one bank per channel), for this system is 18 feet long, 5 feet wide, and 5 feet deep. The overall footprint for installation of this system, including traveling crane, UV disinfection equipment, and peripheral equipment is 32 feet long and 20 feet wide. The manufacturers of UV systems typically provide an automatic level control device to maintain a near constant water surface elevation over the UV lamps. The expected headloss through this system is less than four inches.

### Alternative C - Vertical, Low Pressure System

Alternative C was a vertical, low pressure UV systems. Vertical modules typically consist of 40 lamps, five rows with eight lamps per row. Overall, the dimensions are usually 24-inches wide by 30-inches long. A 12-inch space is required between modules in series. Since the lamp can be accessed from the top, vertical modules do not need to be removed to replace a lamp. Typically, cleaning of the quartz sleeves are performed by removing the entire module and immersing it into a cleaning tank, similar to the conventional low pressure systems.

Vertical, low pressure system for Mount Vernon would include approximately 960 lamps, configured as twenty four 40-lamp modules, for a total of 960 lamps. The modules would be arranged in three channels, with eight modules per channel. The peak power required is 48 kW. This system can be supplied by multiple manufacturers.

The estimated required dimensions for each channel (requires three channels, eight banks per channel), for this system is 40 feet long, 2 feet wide, and 5 feet deep. The overall footprint for installation of this system, including traveling crane, UV disinfection equipment, and peripheral equipment is 62 feet long and 18 feet wide. The manufacturers of UV systems typically provide an automatic level control device to maintain a near constant

water surface elevation over the UV lamps. The expected headloss through this system is less than 4 inches.

### Alternative D - Open Channel, Medium Pressure System

Alternative D was an open channel, medium pressure UV systems composed of a reactor vessel with multiple modules. Modules typically consist of two to eight lamps. The module is designed to raise lamps from the channel to a convenient level outside of the channel for maintenance. Typically, cleaning of the quartz sleeves are performed automatically since fouling of the quartz sleeve occurs rapidly at the operating temperatures.

An open channel, medium pressure system for Mount Vernon would include approximately 48 lamps, configured in one reactor vessel. The reactor would be arranged in one channel. The peak power required is 73.6 kW. This system is proprietary and is supplied by Trojan Technologies.

The estimated required dimensions for the for this system is 36 feet long, 45 inches wide, and 119 inches deep. The overall footprint for installation of this system, including UV disinfection equipment, and peripheral equipment is 44 feet long and 12 feet wide. The expected headloss through this system is one to two feet.

### Alternative E - Closed Conduit, Medium Pressure System

Alternative E included the review of a closed conduit, medium pressure system. For the indicated flow conditions, this system was not cost effective and was eliminated from further consideration.

### Cost Estimates

Capital and operating costs for each alternative was developed for retrofitting the disinfection system in the existing chlorine contact basin. Alternatives A and E are not presented as they were excluded from additional analysis based on their high capital costs alone. The capital costs, summarized in Table 9-7, include a 20% contingency and 30% for indirect project costs. Operations and maintenance costs were based on 20 years at a 5% interest rate.

**Table 9-7**

| Life Cycle Costs (in $1,000) for 25.8 mgd Disinfection Alternatives | | | | | |
|---|---|---|---|---|---|
| **Alternative** | **Description** | **Capital Cost** | **Annual O & M Cost** | **Life Cycle Cost** | **Standby Power Requirements** |
| B | Horizontal, Low Pressure, High Intensity System | $1,500 | $40[1] | $2,000 | 64 kW |
| C | Vertical, Low Pressure System | $1,300 | $37[1] | $1,760 | 96 kW |
| D | Open Channel, Medium Pressure System | $1,340 | $69[1] | $2,200 | 154 kW |

1. Power costs at $0.05 per kWhr

The equipment cost for the low pressure systems, Alternatives B and C, are less expensive than that of the medium pressure system, but due to the maintenance requirements, a building enclosure has been included in the capital cost. The open channel medium pressure system (Alternative D) is a system that is self cleaning and due to the reduced maintenance requirements and system configuration is typically installed without an enclosure. Cost for an enclosure have not been included with this alternative.

Although the life cycle costs are similar, the costs for the medium pressure system are greater than the low pressure systems. The advantage of the medium pressure systems are that due to the greater intensities, they can also be used to disinfect primary effluent. In the case of Mount Vernon, this type of system could also be used for the disinfection of the effluent for the Phase 3 CSO improvements. The medium pressure system can be situated in the existing chlorine contact basins, while providing additional space for a CSO disinfection system. Figure 9-9 presents a preliminary layout of a medium pressure UV disinfection system in the existing chlorine contact basin. The low pressure systems offer higher energy efficiency, but typically require more maintenance since more bulbs are required.

Since the life cycle costs for the vertical low pressure is the lowest and the medium pressure system offers the ability to be compatible with future CSO disinfection requirements, for planning purposes, the capital cost for the medium pressure system has been included. Since medium pressure systems are slightly greater than the low pressure systems, final determination should be made in the design phase.

### UV Design Issues

The micro-organism identified by the NPDES discharge permit affects the design of a disinfection system. Enterococci are more difficult to inactivate than fecal coliform, which results in a larger system, either higher disinfectant dose or longer exposure time. The current NPDES discharge permit is based on fecal coliform. If the regulations change and the permit's basis for compliance is converted to enterococci, then the disinfection system will need to provide additional disinfection capacity. For a UV disinfection system, additional capacity can be easily incorporated through the addition of more UV bulbs to the system.

Redundancy of UV disinfection systems is provided through multiple channels and back-up power generation. Besides the typical redundancy designed in a UV disinfection system, the City of Mount Vernon, should evaluate designing the CSO Treatment Facility's disinfection system to act as a back-up disinfection system during the design phase of the CSO Treatment Facility.

UV disinfection is affected by UV transmittance (UVT), total suspended solids (TSS) concentration, particle size and composition, and wastewater flow rate. UVT is the major parameter used for sizing UV disinfection systems. Upstream processes, industrial dischargers, and the presence of iron compounds may reduce the UVT. Industrial pre-treatment utilizes ferric chloride as a coagulant, which results in the potential for iron to be conveyed to the UV disinfection system. UVT tests performed on the primary effluent and secondary effluent are included as Appendix J. These tests showed lower than expected UVT. Year-round diurnal UVT tests should be performed prior to design, and/or pilot testing of secondary effluent could be utilized to determine the range of UVT. Pilot testing for two (2) months is estimated to cost approximately $30,000.

## SODIUM HYPOCHLORITE SYSTEM

Commercial grade sodium hypochlorite is supplied in a 12.5 percent solution. At 12.5 percent, it rapidly decays (to an 11.0 percent solution in only 30 days). To prevent degradation of the solution, it is recommended that dilution to a 4.0 percent solution occur on site when deliveries are received. Approximately 4,750 gallons of storage would be required to store a month's supply of 4.0 percent solution. In addition to one 5,000 gallon storage tank, ancillary equipment would be required:

- Two 10 gph metering pumps;
- Two 40 gph metering pumps; and
- Three 500 gph transfer pumps.

The sodium hypochlorite system could be located in the existing chlorine feed building, or in a structure adjacent to the existing chlorine facilities. The cost estimate presented anticipates the sodium hypochlorite system will be situated in a room of the existing chlorine facility. A budget of $100,000 has been identified for these improvements.

## EFFLUENT PUMP STATION

The firm pumping capacity of the Effluent Pump Station is 12.0 mgd. Two alternatives were developed to upgraded the Effluent Pump Station to a firm pumping capacity of 25.2 mgd.

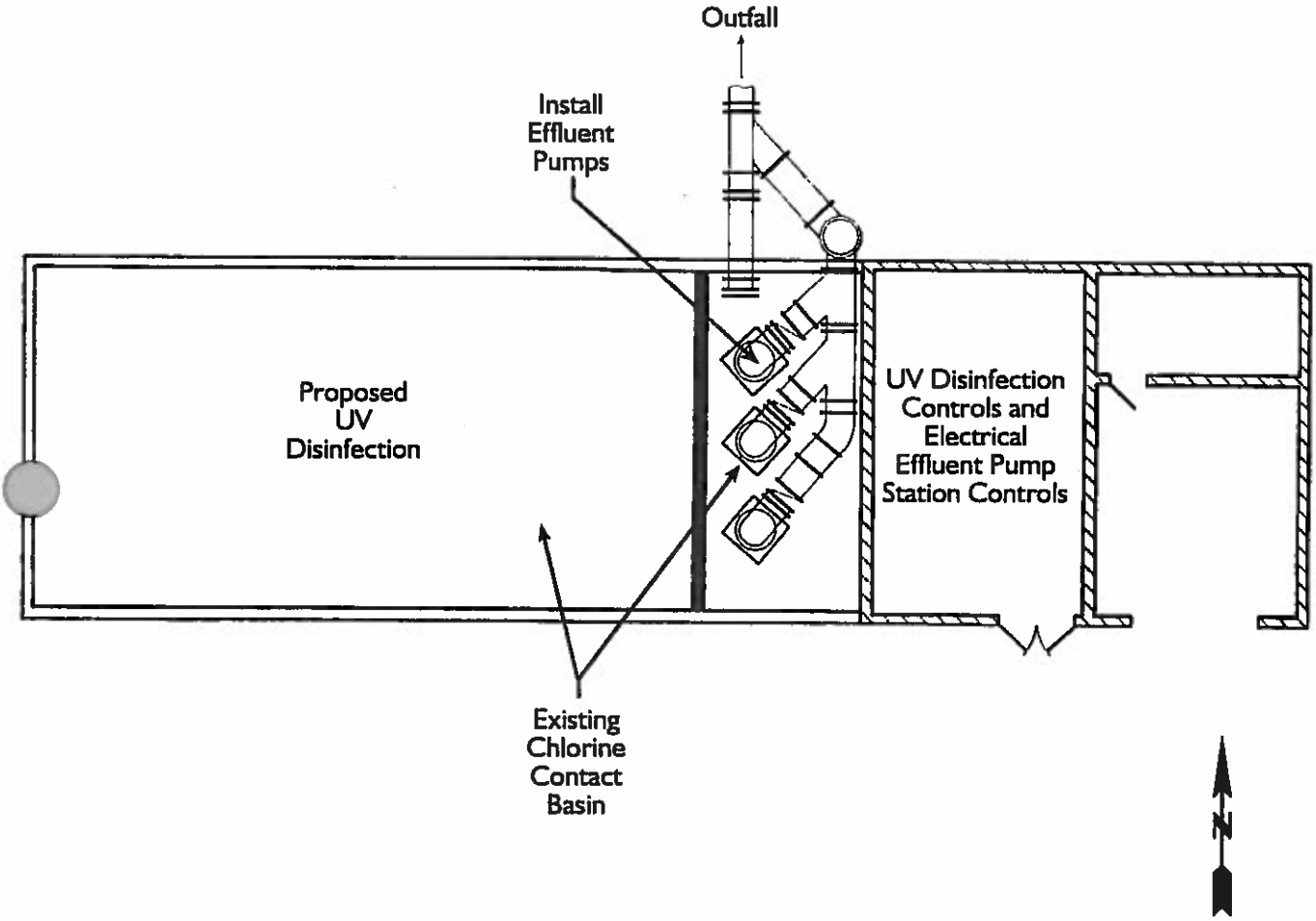
### Alternative A - Retrofit Existing Effluent Pump Station

Alternative A considered retrofitting the existing Effluent Pump Station with new pumps, motors, and controls. The existing Effluent Pump Station is presently equipped with 40 hp pumps. The estimated size for these pumps would be 75 hp. There was not adequate space in the existing facility to install these pumps and this alternative was not considered further.

### Alternative B - Retrofit Existing Chlorine Contact Basin with Effluent Pump Station

Alternative B would retrofit the Effluent Pump Station into the east end of the existing chlorine contact basin. Three pumps would be placed over the chlorine contact basin, utilizing the basin as a wet well. This alternative requires three 75hp pumps, new motors, and controls. It would also require the effluent piping to the outfall to be reconstructed.

A typical plan view of the existing chlorine contact tanks retrofitted with UV disinfection and an Effluent Pump Station is shown on Figure 9-9. The estimated cost for the Effluent Pump Station is $370,000.




HDR Engineering, Inc.

| | |
|---|---|
| Project Title<br>MOUNT VERNON COMPREHENSIVE SEWER | Date<br>FEBRUARY 2003 |
| Sheet Title<br>UV DISINFECTION AND EFFLUENT PUMP STATION | Figure No.<br>9-9 |

## OUTFALL

Alternatives were developed to comply with future flows, loads, and discharge requirements for the outfall. For each alternative, for the secondary treatment, the outfall would terminate in an open ended diffuser at a location near the thalweg (approximately 40 feet farther into the river than the existing outfall), at an invert elevation of approximately -10 feet. This would reduce or eliminate the wastewater from being trapped by near shore eddy currents and improve mixing. An analysis of the mixing zone is presented in Appendix K, Mount Vernon WWTP Mixing Zone Study. The initial requirements for the outfall are as follows:

- Capacity for planned upgrade of the WWTP to a peak hour hydraulic capacity of 25.8 mgd;
- Ultimate capacity for the treated CSO flows (48 mgd peak hour flow, per Alternative 2C, Chapter 4);
- Minimize pumped discharges to high water level conditions in the river; and
- Minimize maintenance requirements.

Two general concepts were reviewed. These included a single outfall for both secondary and treated CSO effluent (Alternative A) and two separate outfall pipes (Alternative B). For preliminary sizing criteria, the velocity of flow within the outfall pipe was limited to 6.0 feet per second. This results in a 48-inch pipe for the single pipe option and 36-inch pipes for the two pipe option. [* Note: As of the finalization of this document, Alternative A was selected and designed]

The flow range from minimum day flow in dry weather conditions of approximately 1.6 mgd to the future peak hour CSO flow of 48 mgd is significant. For the single pipe option, multiple diffusers should be assessed to assure adequate mixing for this large flow range. Based on the recommendations of the Outfall Study, multiple diffusers could present increased maintenance requirements for this river discharge situation.

Cost estimates for the single pipe option (Alternative A) are shown in Table 9-8. Cost estimates for the two pipe option (Alternative B) are shown in Table 9-9

The provisions of a single outfall pipe reduces problems associated with multiple outfalls in close proximity:

1. overlapping mixing zones; and
2. multiple pipes would require additional maintenance.

A summary of the advantages and disadvantages for each alternative is presented in Table 9-10.

Table 9-8

| Single Pipe Outfall (Alternative A) Cost Estimates | | | | |
|---|---|---|---|---|
| **Item** | **Quantity** | **Unit** | **Unit Cost** | **Cost ($1,000)** |
| Sheet Pile | 1 | LS | $250 | $250 |
| Effluent Pipe | 1 | LS | $250 | $250 |
| Outfall Pipe | 1 | LS | $300 | $300 |
| Subtotal | | | | $800 |
| Contingency (20%) | | | | $160 |
| Indirect Project Costs (30%) | | | | $240 |
| Total | | | | $1,200 |

Table 9-9

| Two Pipe Outfall (Alternative B) Cost Estimates | | | | |
|---|---|---|---|---|
| **Item** | **Quantity** | **Unit** | **Unit Cost** | **Cost ($1,000)** |
| Sheet Pile | 1 | LS | $250 | $250 |
| Effluent Pipe | 2 | LS | $200 | $400 |
| Outfall Pipe | 2 | LS | $250 | $500 |
| Subtotal | | | | $1,150 |
| Contingency (20%) | | | | $230 |
| Indirect Project Costs (30%) | | | | $345 |
| Total | | | | $1,725 |

Table 9-10

| Outfall Alternative Advantages and Disadvantages | | |
|---|---|---|
| **Alternative** | **Advantages** | **Disadvantages** |
| Single Pipe | • Lower Capital Cost<br>• Single dilution zone | • Combined effluents would need to be addressed in future NPDES permits |
| Two Pipe | • Maintains option for separate CSO outfall<br>• Lower maintenance (diffusers not required) | • Greater Capital Cost<br>• Multiple dilution zones in close proximity |

The single pipe option is recommended. It has the advantage of a lower capital cost and results in only one dilution zone for both the treated CSOs and the secondary effluent.

### DISSOLVED AIR FLOATATION THICKENER

The existing DAFT is provided for WAS thickening and has an area of approximately 1300 SF. An additional 750 SF is required to treat 2020 WAS flows without nitrication (i.e. BOD removal only), and approximate 500 SF with nitrification. However, this estimation is based on the maximum solids loading rate of 2.5 lb/SF/hour from Department of Ecology for WAS thickening with coagulant/polymer. It would be more conservative design a new system at a lower solids loading rate of 2 lb/SF/hour

The existing DAFT is sized to adequately thicken WAS flows through 2009 with nitrification. A new DAFT would be requied by 2009 with or without nitrification. Using the solids loading rate of 2.0 lb/SF/hour, an additional 40-FT diameter unit would be required by the year 2009 to meet the flows from 2009 through 2020. The new unit will be the same size as the existing unit.

The existing solids process equipment is located in the northeast portion of the WWTP site. Location for a future DAFT has been designated between the digester complex and the Influent Pump Station.

### ANAEROBIC DIGESTER

An additional digester should be provided to reduce the difficulties associated with cleaning the existing digester. It would provide redundancy and allow existing tankage used for storage of solids to be converted to CSO storage, further reducing overflows. A new digester should be sized similar to the existing digester. The estimated cost of a new 103,400 cf digester is $2,500,000.

The existing anaerobic digester is located in the northeast portion of the WWTP site. Location for a future anaerobic digester has been designated between the digester complex and the Influent Pump Station. This is the logical location for a future anaerobic digester.

## ENERGY RECOVERY

Methane gas is a byproduct of the anaerobic digestion process. Currently, the plant produces approximately 30,000 cubic feet (cf) per day. A portion of this gas is used to heat the incoming sludge, and the remainder is flared. Historically, power generation from waste gas was accomplished with internal combustion engines and generators. Due to the minimum sizing requirements for the engine generator units and relatively low electrical power costs, the generation of energy from waste digester gas has been historically limited to facilities much larger than the Mount Vernon WWTP. Based on plant estimates, the quantity flared is approximately 50% of the gas production. Based on a value of 650 BTU per scf, the average amount of waste digester gas currently flared is 10 MBTU per day. This equates to approximately 50 hp, or 37 kW.

In recent years, power costs have increased and there are now newer technologies available for electrical power generation. In addition to conventional internal combustion engine generator units, small turbine units (microturbines) are available.

Another emerging technology is the use of fuel cells. These devices convert hydrogen into electrical power and water. Fuel cell technology for wastewater treatment plants is still in the development phase. Fuel cell technology may become cost effective for the Mount Vernon WWTP in the future, but at this time it is not recommended for consideration.

Another recent technology for cogeneration is the use of microturbines (see Appendix L). Current units are available with capacitles of 30 kW. This smaller incremental size creates opportunities for intermediate sized WWTPs to more cost effectively generate electrical power from waste digester gas. Since the WWTPs minimum electrical demand would be less than the capacity of the units, the electrical intertie would be simplified and would operate in a 'grid connect' mode. A preliminary estimate was completed for the installation of a microturbine cogeneration facility at the plant. Three size increments were considered, 30, 60, and 90 kW. The unit would be located adjacent to the Solids Handling Building. The units would be provided with a roof structure. Preliminary cost estimates were developed for 30, 60, and 90 kW facilities as presented in Table 9-11.

Table 9-11

| Co-generation with Microturbines Cost Estimates | | | |
|---|---|---|---|
| | Co-generation Capacity | | |
| Item | 30 kW | 60 kW | 90 kW |
| Capital Cost | $170,000 | $300,000 | $390,000 |
| Annual Debt Recovery[1] | $14,000 | $24,000 | $31,000 |
| Debt Recovery/kWhr[2] | $0.06 | $0.05 | $0.04 |
| Maintenance Cost/kWhr[3] | $0.03 | $0.02 | $0.02 |
| Total Power Cost | $0.09 | $0.07 | $0.06 |

1. 20 years, interest 5%
2. Based on 90% operating time
3. Includes cost to rebuild unit at 40,000 hrs

Current electrical energy costs average $0.05 per kWhr and preliminary estimates of energy available from the waste digester gas is 40 kW. Depending on interest rates for payback on the capital cost, at this time, it may not be cost effective for the City to install this type of system. Factors that could make this type of system cost effective include:

- Increased electrical energy costs;
- Increased loads to the WWTP and related digester gas production; and
- Available funding (with grant monies to assist with capital cost, the system could be cost effective at current conditions).

### ODOR CONTROL

Gas-stream odor control at the WWTP can be accomplished through collection of odorous gases and treatment with scrubbers. Collection of odorous gases occurs through containment or covering unit processes. Containment can be accomplished with a building, such as a headworks building. Covering can be performed with either concrete, aluminum, plastic, or fiberglass, such as covers over the primary clarifiers or influent pump station wet well. Gas-phase odors are collected and treated in one of numerous unit processes: biofilters, chemical scrubbers, packed-bed wet scrubbers, mist scrubbers, or carbon absorbers. The most economical solution for a plant the size of Mount Vernon is typically collection of gases through a combination of covers and containment and treatment with a wet scrubber. An estimated cost for such a system (covers on the primary clarifiers and grit

basins, containment of odors in the Influent Pump Station and Headworks building, and treatment with a wet scrubber) is $1,300,000, as presented in Table 9-12. Additional unit processes can be covered to contain all potential odors.

**Table 9-12**

| Odor Control Cost Estimate | | | | |
|---|---|---|---|---|
| Item | Quantity | Unit | Unit Cost | Cost |
| Site Preparation | 1 | LS | $35,000 | $35,000 |
| Primary Clarifier Covers[1] | 10,800 | SF | $40 | $432,000 |
| Grit Basin Covers[1] | 630 | SF | $40 | $25,200 |
| Duct Work | | LS | $141,500 | $141,500 |
| Packed Tower - Wet Scrubber | | LS | $250,000 | $250,000 |
| | | | | |
| Subtotal | | | | $883,700 |
| | | | | |
| Contingency (20%) | | | | $176,700 |
| Indirect Project Costs (30%) | | | | $265,100 |
| | | | | |
| Total | | | | $1,325,500 |

1. Covers include influent and effluent channels and structure

## BIOSOLIDS REQUIREMENTS

Subpart D (pathogen and vector attraction reduction) requirements of the 40 CFR Part 503 regulation apply to sewage biosolids, both bulk biosolids and biosolids that is sold or given away in a bag or other container for application to the land, and domestic septage applied to the land or placed on a surface disposal site. There are two basic types of requirements in Subpart D, Class A and Class B. Class A requirements are to reduce biosolids pathogens to below detectable levels. Class B requirements are to ensure that pathogens have been reduced to levels that are unlikely to pose a threat to public health and the environment under the specific use conditions. Regulations also require a reduction in the potential of biosolids to attract vectors, such as rodents, birds, insects, and other organisms that can transport pathogens.

### Class B Biosolids

Mount Vernon currently treats the biosolids from the WWTP to Class B standards. Permitting for land application has not been a problem. At this time, there does not appear to be a need to increase the treatment level to Class A. If the situation would change and land application sites were not available, then the City may want to consider providing Class A biosolids.

### Class A Biosolids

Mount Vernon is not required to produce Class A Biosolids, but if they chose to treat the biosolids to this level, it can be met by any of the following processes:

- Biosolids can be thermally treated by using a specific time-temperature regime to reduce pathogens. One option is to use a heat drying system to provide heat treatment of the digested dewatered material. With this process, a dewatered biosolids cake enters a heat drying system where thermal energy is added for the evaporation of entrained water. The biosolids are dried to a solids concentration of from 90 to 96 percent and the end product is in the form of a dried pellet. These pellets can then be used as fertilizer. In addition to the capital cost for the system and labor requirements, a large amount of energy is required to dry the biosolids. Based on typical thermal efficiency of the systems, approximately 1,500 BTUs per pound of water evaporated is required. Starting with a solids concentration of 16 percent and drying it to 95 percent would require approximately 16 million BTUs per dry ton of solids. At a cost of $0.90 per Therm for natural gas, this would equate to an energy cost of approximately $150 per dry ton of solids. Allowing a capital cost of $100 per dry ton and a labor cost of approximately $50 per dry ton would result in a total cost of approximately $300 per dry ton for biosolids handling. Based on these costs, this alternative is one of the higher cost options for obtaining Class A biosolids.

- High temperature-high pH treatment is the process also known as alkaline treatment. It exposes biosolids to pHs greater than 12 for greater than 72 hours, and simultaneously has temperatures greater than 52 degrees Celsius for over 12 hours. Air drying is the last step of the process. Drying is performed to provide a solids concentration of greater than 50 percent after the 72 hours of pH-temperature treatment. The unit cost for this process is typically $200 to $250 per dry ton.

- Composting requirements vary depending on the composting process chosen. For an aerated static pile, the temperature must be maintained above 55 degrees Celsius for greater than 3 days. For a windrow composting method, the temperature must be maintained above 55 degrees Celsius for greater than 15 days, with a minimum of five turnings of the windrow. The unit cost for this process is typically $125 to $175 per dry ton.

If the City were to decide in the future to treat biosolids to Class A standards, the recommended option would be to utilize aerated static composting. This has been used by a number of similar sized communities. The advantages are that it is a relatively simple process to maintain and the end product is Class A biosolids, which has a relatively high demand. This unit process would require a capital investment of $860,000 and an annual

O&M cost of $150,000. These costs are above and in addition to the current capital and operation and maintenance costs required in the other sections of this Comprehensive Sewer Plan Update.

## FACILITIES

### Operations Building

The Existing Administration/Laboratory Building is limited in space for the current operations. The existing laboratory is located within this building, along with the lunch room, lockers/showers and office space. This laboratory is adequate for current needs, but should eventually be expanded.

Based on discussions with City staff, it may be desirable to provide a phased approach to meet future operations building and laboratory requirements. Initialy a new Wastewater Utility Administration Building would be constructed. This would include the following:

- Reception area
- Office space
- Meeting rooms
- Lunch Room
- Mens locker/shower
- Womens locker/shower
- Library

At the same time the the existing Administration/Laboratory building would become the Laboratory/Operations Center. This would include the following:

- Laboratory (no changes to existing laboratory)
- The remainder of the building would become the Operations Center and would include:
    - Operator work areas
    - SCADA system monitoring
    - Plan/Map storage
    - Deliveries

- Library

Preliminary total project cost for the initial initial phase is $600,000.

Additional budget should be provided for long term planning to provide an upgrade of the existing laboratory. At that time the existing Laboratory/Operations Center could be converted to all laboratory facilities and additional Operations Center facilities provided. A preliminary budget of $600,000 for this long term improvement.

**Shop/Garage**

The existing shop and garage will need to be dedicated to the WWTP in the future. This will necessitate construction of a new garage/vehicle storage building for the collection system equipment and the grounds maintenance equipment. This building should contain five vehicle bays and an area dedicated to maintenance. It should be a minimum 4,000 sf to accommodate the vehicle storage and maintenance. An estimated cost for a 4,000 sf shop/garage is $500,000. based on discussions with plant staff, the primary need for this building is for material and vehicle storage and if required to reduce the cost, a "carport" type covered structure could be provided.

## RECLAIMED WATER FEASIBILITY

### Background

The City of Mount Vernon reviewed the feasibility of wastewater reclamation in its service area. Potential end uses for reclaimed water include urban and agricultural irrigation, and less common applications such as wetland creation, and direct or indirect streamflow augmentation. Table 9-13 lists the anticipated water quality objectives for various potential reclamation end-uses.

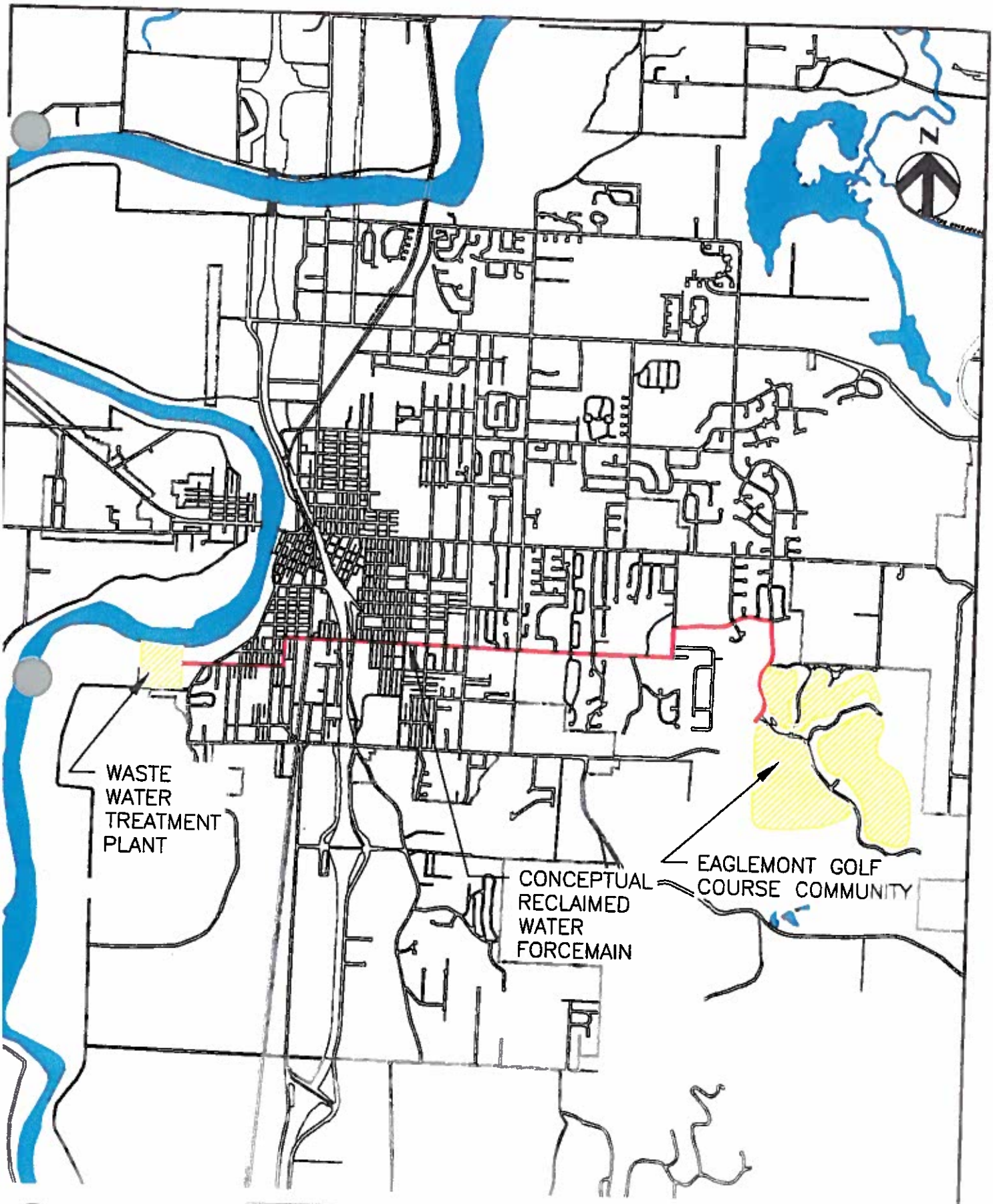
**Table 9-13**

**Water Quality Classifications for Reclamation End-Uses**

| Water Quality | BOD mg/L | TSS mg/L | Total P mg/L | NH3-N mg/L | TN mg/L | Turb. NTU | TOC mg/L | TDS mg/L | Metals, Organics |
|---|---|---|---|---|---|---|---|---|---|
| Class A | 30 | 30 | -- | -- | -- | 2 | -- | -- | -- |
| Wetlands | 20 | 20 | 1 | Toxicity | 3 | 2 | -- | -- | Surface2 |
| GW (perculation) | 30 | 30 | -- | -- | 10 | 2 | -- | -- | Site |
| GW (non-potable) | 5 | 5 | -- | -- | Site | 2 | -- | Site | Site |
| GW (potable) | 5 | 5 | -- | -- | 10 | 0.1 | 1 | Site | SDWA |
| Large Stream (marine) | 30 | 30 | 3-5 | 2-3 | -- | 2 | -- | -- | Surface1 |
| Small Stream (marine) | 10 | 10 | 1-2 | 1 | -- | 2 | -- | -- | Surface1 |
| Large Stream (lake) | 30 | 30 | 0.1 | 2-3 | -- | 2 | Pos | -- | Surface1 |
| Small Stream (lake) | 10 | 10 | 0.1 | 1 | -- | 2 | Pos | -- | Surface1 |
| Lake Anticipated | 10 | 10 | 0.01 | 1 | -- | 2 | -- | 500 | SDWA |
| Lake Worst Case | 10 | 10 | 0.01 | 0.02 | 0.6 | 2 | 2 | 100 | SDWA/BG |

Notes:
GW = Groundwater recharge
Pos = Possible limit
Site = Site specific criteria
Surface1 = Surface water standards with mixing zone
Surface2 = Surface water standards with mixing zone
SDWA = Drinking water standards
BG = Background concentrations without mixing zone

At a minimum, the reclaimed water treatment processes must meet Class A water quality standards for oxidation, filtration, and disinfection. Depending on the end uses, additional treatment could be required to meet more stringent nutrients, metals, organics, and turbidities levels.




HDR Engineering, Inc

Project Title
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date
FEBRUARY 2003

Sheet Title
CONCEPTUAL RECLAIMED WATER FORCEMAIN ALIGNMENT

Figure No
9-10

### Potential Customer

The Eaglemont Golf Course Community located in SE Mount Vernon was identified as a potential customer for reclaimed water. The Eaglemont community plan encompasses 675 acres. Nearly 60% of this acreage is committed to open space, including the golf course and wetlands, two mini-parks and a five-acre neighborhood park. A beaver pond and nature preserve account for another 30 acres. Reclaimed water could be used to satisfy the irrigation water need, and potentially for maintaining the existing wetlands and ponds.

### Treatment Processes Required

The existing Mount Vernon wastewater treatment plant consists of primary treatment, secondary activated sludge system for BOD removal, and disinfection. In order to provide the level of treatment to produce reclaimed water, additional treatment processes for turbidity reduction and additional disinfection would be required. The turbidity reduction would be accomplished by a filtration step utilizing multimedia sand filtration, or membrane filtration such as microfiltration. A separate disinfection process via the ultraviolet (UV) process to meet the reclaimed water standards.

A new reclaimed water pump station and a new force main, approximately 4 miles long, would be required to deliver reclaimed water from the existing wastewater treatment plant to the Eaglemont community. Figure 9-10 shows the proposed conceptual alignment of the reclaimed water forcemain.

The current irrigation water use at the Eaglemont community in the irrigation season is estimated to be 1 MGD on average. A conceptual level cost estimate was developed gpt s 1 mgd reuse plant. Table 9-14 summarizes the capital cost of the conceptual level new reclaimed water treatment system and related distribution infrastructure.

**Table 9-14**

**Estimated Capital Cost of 1 MGD Reclaimed Water Treatment System and Distribution Infrastructure**

| Component | Capital Cost |
|---|---|
| Membrane Bioreactor for nutrient removal and membrane filtration | $1,000.000 |
| UV Disinfection System and Pump Station | $500,000 |
| Forcemain | $2,000,000 |
| Subtotal | $3,500,000 |
| Tax (8%) | $280,000 |
| Contingency (35%) | $1,225,000 |
| Total | $5,000,000 |

## Feasibility of Implementing Water Resue in the City of Mount Vernon

At present, the portion of water flow from the municipal supply system used for irrigation in the City of Mount Vernon would not be returned to the Skagit River. If reclaimed water was available for irrigation, the amount of municipal water demand could be reduce proportionally, thereby reducing the diversion of freshwater from the river.

Based on this conceptual cost estimate, using reuse water is not cost effective compared to the use of current muncipal water supply. The higher cost of reuse water is associated with the capital cost of building the new advance watewater treatment facilities and constructing the distribution infrastructure, and the operation and maintenance of such a system. At this time, this is not economically favorable to implement water reuse.

# 10. RECOMMENDED WWTP ALTERNATIVES

This chapter presents the recommended alternatives for upgrade to the existing WWTP.

## HYDRAULICS

The alternate WWTP hydraulics is recommended (Alternative B). The advantages include easier access to equipment and pumping forward flows only once. With selection of Alternative B, the existing activated sludge pump station could be designated entirely for RAS pumping.

## INFLUENT PUMP STATION

### Pump Station Capacity

Alternative A is the preferred alternative. The existing pump station can be retrofitted with new pumps and motors for approximately $0.6 million less than utilizing submersible pumps. The pump station should be upgraded to 24 mgd with four 75 hp pumps and motors for an estimated cost of $1.6 million. A physical model of the pump station, before and after conditions, should be considered during the pre-design phase to assure that current problems are corrected by the improvements.

### Screening

Coarse screening, with 1-inch screen spacing, is recommended to provide protection for the influent pumps. The existing manually-cleaned bar screen should be replaced with a mechanically-cleaned screen, and the existing mechanically-cleaned screen should be utilized as a backup unit. The estimated cost for replacing the manually-cleaned bar screen with a mechanically-cleaned screen is included in the cost estimate of upgrading the influent pump station, see above.

## HEADWORKS

A headworks facility would improve the screening and grit removal, protecting downstream process equipment. The estimated cost of a headworks facility is $2.8 million. Details of the recommended headworks are discussed below.

### Comminutor

The comminutor is recommended for abandonment.

### Fine Screens

Installation of fine screens is recommended. Fine screens should have 3/8-inch spacing and be mechanically-cleaned and provided with washing and compacting equipment.

### Grit Removal

A vortex grit removal system is recommended because of the high flexibility coupled with moderate costs. The hydrocyclone de-gritter has both a high capital and operating cost. The aerated grit chamber has low flexibility and a high operating cost.

### Disposal

The existing method of disposal is recommended to be continued. It also is recommended that a building be placed around the screenings and grit storage site to prevent unpleasant odors from escaping the site.

## PRIMARY CLARIFIERS

Two new (75-foot diameter) clarifiers are recommended. The life cycle costs of the alternatives are relatively equivalent. The two new clarifiers offers advantages that off-sets the minimal cost difference seen over the life of the clarifier. These advantages include:

- Reserves capacity of the existing clarifier for combined sewer flows (for the 'internal shunt');
- Construction cost savings may be realized, as construction sequencing will be less than the cost towhen to modify the existing clarifier; and
- Two clarifiers would provide redundancy for regular maintenance and unexpected circumstances.

The estimated cost of two new clarifiers is $1.8 million.

## ACTIVATED SLUDGE PROCESS

The existing activated sludge system is recommended to be converted from the existing BOD removal mode to a nitrification mode. This conversion will necessitate additional aeration basin capacity and blowers. Details of all recommended improvements for the activated sludge process are below:

### Activated Sludge Pump Station

The existing activated sludge pump station is recommended to be designated as an RAS pump station. It has 24.0 mgd capacity (firm pumping capacity of 16.0 mgd), which is in excess of 100 percent of the forward flow through the secondary process at 2020 (9.9 mgd).

### Selector Basin

A selector basin is recommended for filament control. A selector basin will allow filamentous bulking control without the use of chlorine. It can be constructed adjacent to the RAS pump station and as detailed in Alternative B. This could be constructed in two phases, the second phase incorporated with the addition of nitrification. The total estimated cost for this selector basin is $600,000.

### Aeration Basin

Alternative B, nitrification mode, is required to meet anticipated NPDES permit limits, based on the TMDL of the Skagit River and the toxicity testing (which will most likely limit the allowable ammonia concentration). This alternative utilizes the 0.5 mg Aeration Basin No. 4, requires an additional 1.2 mg aeration basin volume, and replaces the coarse bubble diffusers with fine bubble diffusers. The estimated cost for these improvements is $2.7 million, and could be performed in a phased manner over the 20-year planning horizon.

### Blowers

Addition of one blower by 2020 is recommended. The existing blowers have capacity to meet aeration requirements until 2010. One additional blower will meet aeration requirements through 2020. The estimated cost of improvements (building expansion, piping modifications, and one additional blower) are estimated at $333,000.

### Secondary Clarifiers

The existing Secondary Clarifier No. 1 (peripheral feed clarifier) is recommended for conversion to WAS storage (aerobic digester). By moving the WAS storage from Aeration Basin No. 4 to the inefficient Clarifier No. 1, it opens up aeration basin volume and reduces the additional aeration basin volume required. It also removes an inefficient secondary clarifier, and replaces it with an efficient clarifier.

It is recommended that three additional clarifiers be added. Two clarifiers should be on line by 2010. One clarifier should be on line by 2020. The estimated costs for 2010 are $2.5 million and for 2020 are $1.1 million.

## DISINFECTION

Alternative C, a vertical, low pressure UV disinfection system, has the lowest life cycle cost. It is recommended to replace the existing chlorine disinfection system. While the low pressure UV system is the least costly alternative, there may be advantages to utilizing a medium pressure system, such as locating the CSO treatment disinfection system, secondary effluent disinfection system, and effluent pump stations in the existing chlorine contact basin. The budgetary cost estimate, $1.34 million, for this planning level determination has been estimated as the higher of the costs ($1.30 million for low pressure verses $1.34 million for medium pressure) for a UV disinfection system and will allow the most beneficial disinfection system to be chosen during the design phase.

## SODIUM HYPOCHLORITE SYSTEM

A sodium hypochlorite system is recommended to provide chlorine for miscellaneous plant uses. The description of system equipment is presented in Chapter 9. The hypochlorite system's transfer and metering pumps, and storage tank (5,000 gallon) could be located in the existing chlorine room. Ventilation requirements and compliance with Article 80 of the Uniform Fire Code will need to be assessed when utilizing the existing chlorine room. The estimated cost for a sodium hypochlorite system is $100,000.

## EFFLUENT PUMP STATION

It is recommended that the existing effluent pump station be abandoned. The existing pump station can be converted to contain the electrical and controls for the UV disinfection system and the proposed effluent pump station.

A new pump station, Alternative B, consisting of low head pumps, can be incorporated into the existing chlorine contact basin. The downstream portion of the contact basin could be utilized as the wet well of the pump station, and configured to flow by gravity to the outfall under normal operating conditions. The pump station would consist of three low head pumps, with a firm pumping capacity of 25.8 mgd. The actual sizing of the pumps will depend on the design of the outfall, but preliminary sizing estimates 75 hp pumps. The estimated cost for this pump station is $370,000.

## OUTFALL

The recommended outfall improvement is Alternative A. It promotes better dispersion than the existing outfall and maintains effluent flows away from the near shore Eddies. The estimated cost of replacing the outfall, including the piping from the WWTP, is $1,200,000.

## DAF THICKENER

A new DAFT is recommended to meet the year 2020 loadings. A 40-ft-diameter unit will provide capacity for loadings through 2020. The details for this recommendation are presented in Chapter 8 , and the cost is estimated to be $400,000.

## ANAEROBIC DIGESTER

A new anaerobic digester is recommended to provide redundancy and digester volume while cleaning the existing digester. A 60-ft-diameter unit with a sidewater depth of 34-feet would be adequate to meet redundancy and flow requirements through 2020. The cost is estimated to be $2,500,000.

## ODOR CONTROL

It is recommended that gas-phase odors be treated at the WWTP. Odors (gas-phase) should be collected from above the influent pump station wet well, headworks building, and primary clarifiers. The gas-phase odors could be treated with wet scrubber and discharged to the atmosphere. The estimated cost for gas-stream treatment of odors by collection and a single scrubber is $1,300,000.

## BIOSOLIDS REQUIREMENTS

It is recommended that Mount Vernon continue to treat biosolids to Class B standards. If Mount Vernon were to treat biosolids to Class A standards, it would be recommended to utilize aerated static composting, at a capital investment of approximately $860,000 and an annual operation and maintenance cost of $150,000.

## FACILITIES

### Operations Building

A new Operations Building is recommended as a first phase improvement, at an estimated cost of $500,000. During predesign, details the final requirements should be confirmed and the final budget refined.

### Shop/Garage

Addition of 4,000 sf of garage/vehicle storage is recommended, at a cost of $500,000. During predesign, details, such as the square feet of garage space, additional shop space, etc. should be determined.

## SITE IMPROVEMENTS

### 100-year Flood Protection

The existing dike between the WWTP and the Skagit River will protect the WWTP from inundation of the 25-year flood event (estimate based on conversations with the ACOE). Flows in excess of the 25-year flood event will most likely result in a failure of the existing dike downstream of the WWTP. Backwater affects will result of inundation of the WWTP to a water surface elevation of 28.2-28.3 ±0.5 ft. To provide protection from the 100-year flood event, the WWTP should consider construction of a dike around the entire plant. The estimated costs for a 2,000 LF ring dike are $600,000, including 20% contingency and 30% indirect costs. Actual costs will vary depending upon the necessary site improvements.

### Roadways

Modification of the existing WWTP will include construction of new process equipment, modification of old process equipment, and new facilities. Improvements to the site should also be planned for, such as re-routing existing roadways or construction of new roadways. It is estimated that 1,300 LF of new roadway will be required at an estimated cost of $50,000, including 20% contingency and 30% indirect costs. Actual costs will vary depending upon the necessary site improvements.

### Drainage

Modification of the existing WWTP will also require improvements to the drainage on site. It is estimated that 11 acres of area will be modified requiring new drainage. An estimated cost of $250,000, including 20% contingency and 30% indirect costs has been budgeted for drainage improvements. Actual costs will vary depending upon the necessary site improvements.

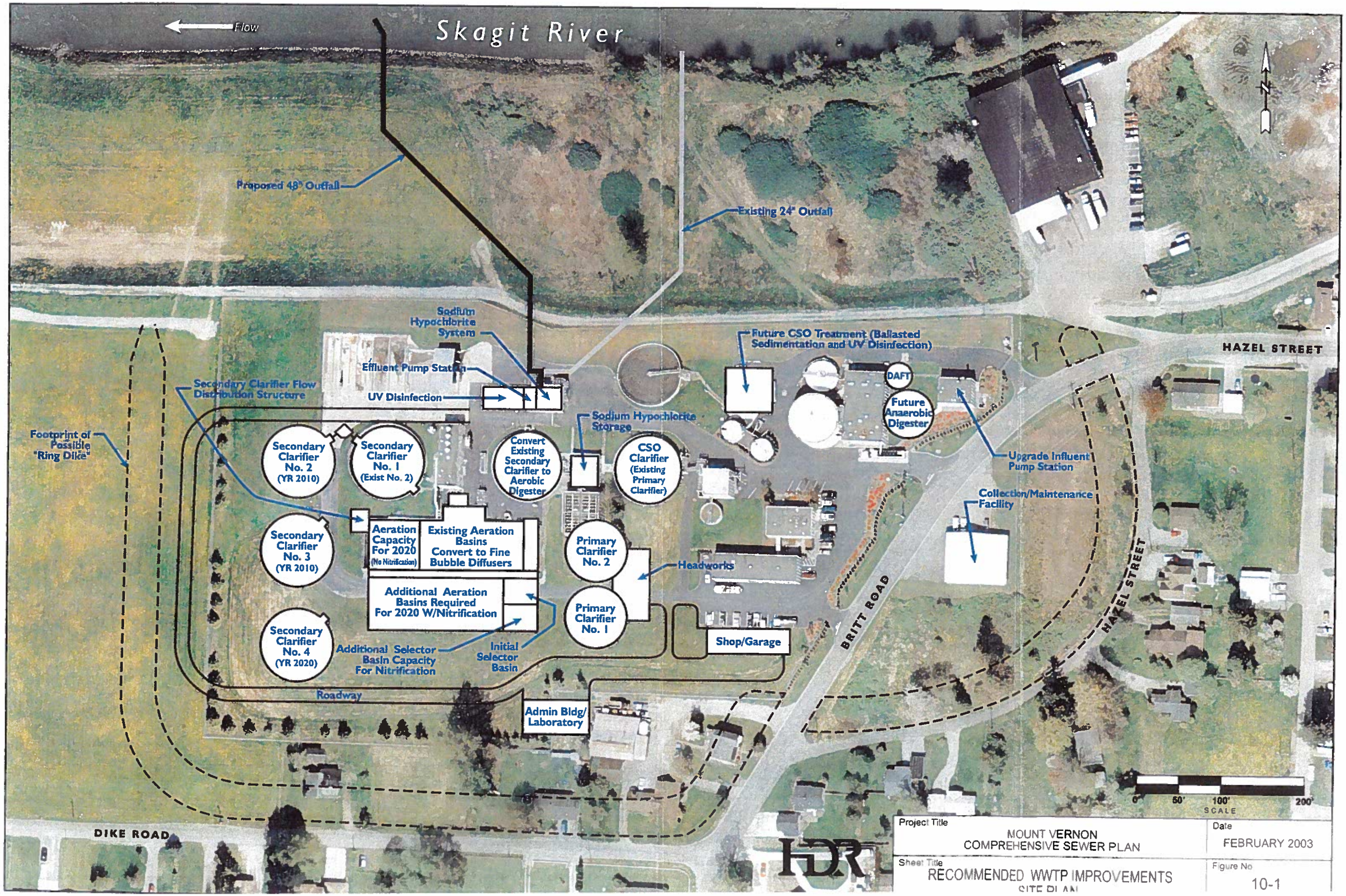
## SUMMARY OF RECOMMENDATIONS

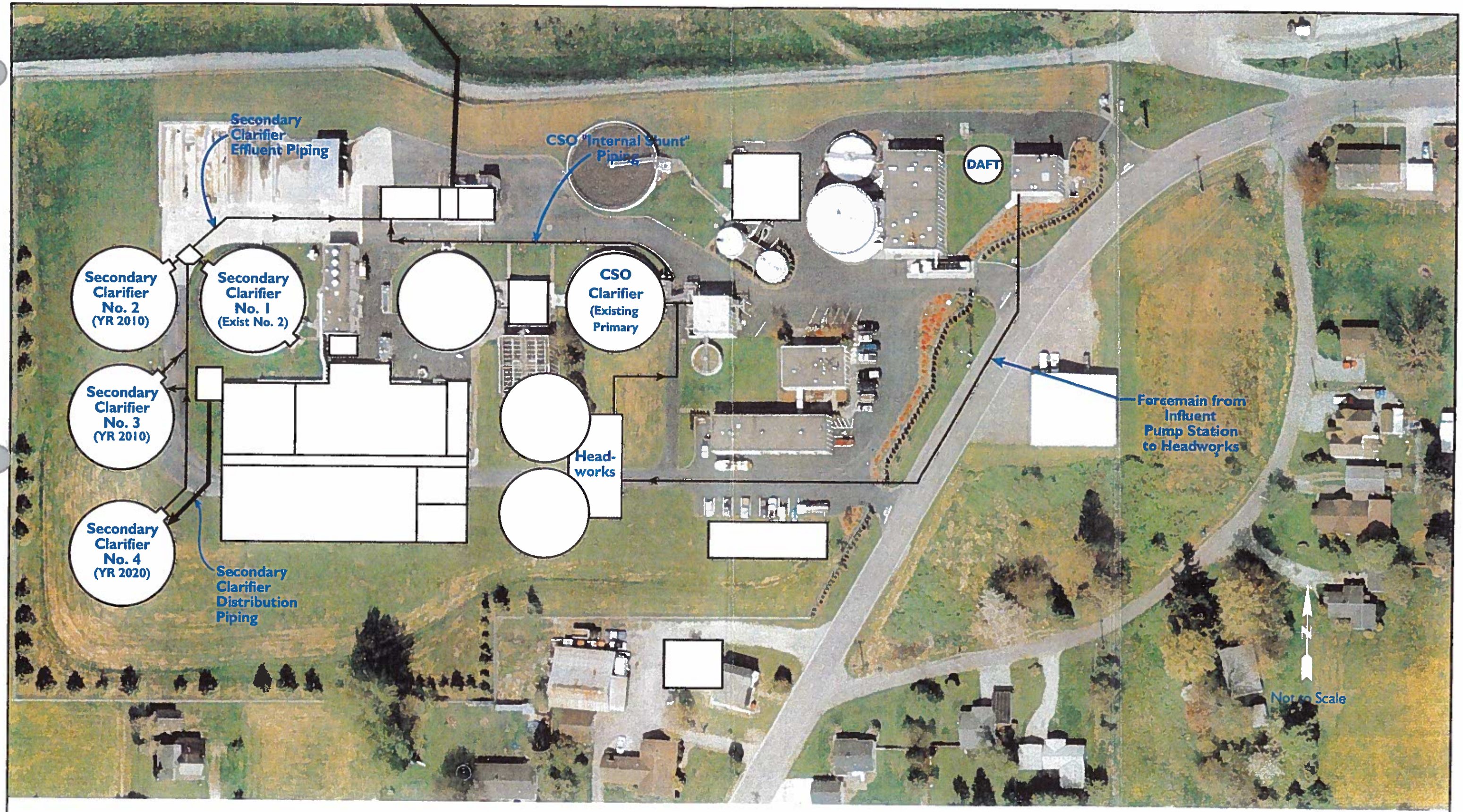
Table 10-1 presents a summary of the recommended improvements and cost estimates for each WWTP improvement. Table 10-2 presents a summary of the recommended improvements and cost estimates for each CSO Treatment improvement.

**Table 10-1**

| Recommended Improvements for the Wastewater Treatment Plant | |
|---|---|
| **Improvement** | **Capital Cost Estimate ($1,000)** |
| Influent Pump Station | $1,600 |
| Headworks | $2,800 |
| Primary Clarifiers | $1,800 |
| Selector Basins | $600 |
| Aeration Basins | $2,700 |
| Chemical Feed System (pH control) | $50 |
| Secondary Clarifiers | $3,600 |
| UV Disinfection[2] | $1,340 |
| Effluent Pump Station | $370 |
| Outfall | $1,200 |
| Sodium Hypochlorite System | $100 |
| DAFT | $400 |
| Anaerobic Digester | $2,500 |
| Odor Control System | $1,300 |
| Administration Building | $500 |
| Laboratory Expansion/Operations Center | $600 |
| Shop and Garage | $500 |
| Flood Protection - 100-year event | $600 |
| Roadways | $250 |
| Drainage Improvements | $50 |
| Total | $23,593 |

1. ENR Construction Cost Index 6397, October 2001.
2. UV disinfection costs include capital cost of a UV disinfection system and costs for pilot testing for two months.


Flow
Skagit River
Proposed 48" Outfall
Existing 24" Outfall
Sodium Hypochlorite System
Effluent Pump Station
UV Disinfection
Secondary Clarifier Flow Distribution Structure
Footprint of Possible "Ring Dike"
Secondary Clarifier No. 2 (YR 2010)
Secondary Clarifier No. 1 (Exist No. 2)
Convert Existing Secondary Clarifier to Aerobic Digester
Sodium Hypochlorite Storage
CSO Clarifier (Existing Primary Clarifier)
Future CSO Treatment (Ballasted Sedimentation and UV Disinfection)
DAFT
Future Anaerobic Digester
Upgrade Influent Pump Station
Collection/Maintenance Facility
HAZEL STREET
Secondary Clarifier No. 3 (YR 2010)
Aeration Capacity For 2020 (No Nitrification)
Existing Aeration Basins Convert to Fine Bubble Diffusers
Primary Clarifier No. 2
Headworks
Additional Aeration Basins Required For 2020 W/Nitrification
Primary Clarifier No. 1
Secondary Clarifier No. 4 (YR 2020)
Additional Selector Basin Capacity For Nitrification
Initial Selector Basin
Shop/Garage
BRITT ROAD
Roadway
Admin Bldg/ Laboratory
DIKE ROAD
0' 50' 100' 200'
SCALE
HDR
Project Title
MOUNT VERNON
COMPREHENSIVE SEWER PLAN
Date
FEBRUARY 2003
Sheet Title
RECOMMENDED WWTP IMPROVEMENTS
SITE PLAN
Figure No
10-1




HDR Engineering, Inc.

| | |
|---|---|
| Project Title<br>MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE | Date<br>FEBRUARY 2003 |
| Sheet Title<br>RECOMMENDED YARD PIPING | Figure No<br>10-2 |

# 11. Capital Improvement Plan

This chapter presents a summary of the improvements for the City of Mount Vernon as a plan for improvement and expansion. Improvements for the combined sewer system, CSO reduction were developed in Chapter 4. Improvements for the wastewater collection system were developed in Chapter 5. Improvements for the wastewater treatment plant were developed in Chapter 10.

## Capital Improvement Schedule

A capital improvement schedule is based on improvements necessary for future CSO reduction, collection system improvements and expansion, and wastewater treatment plant improvements and expansion. Table 11-1 presents the recommended capital improvement schedule for the Wastewater Treatment Facility. Table 11-2 presents the recommended capital improvement schedule for CSO Treatment. Table 11-3 presents the recommended capital improvement schedule for the collection system. Table 11-4 presents a summary of all system improvements.

Table 11-1

| Table 11-1 WWTP Capital Improvement Schedule 2000-2020 ($1,000) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Improvement | 2001 | 2002 | 2003 | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | >2015 |
| Influent Pump Station | | | | 1,600 | | | | | | | | | | | |
| Headworks | | | | 2,800 | | | | | | | | | | | |
| Primary Clarifier | | | | 1,800 | | | | | | | | | | | |
| Activated Sludge - Fine Bubble Diffusers | | 300 | | | | | | | | | | | | | |
| Activated Sludge - Selector Basin | | | | 300 | | | | | 300 | | | | | | |
| Activated Sludge - Chemical Feed System (pH control) | | 50 | | | | | | | | | | | | | |
| Activated Sludge - Additional Aeration Basin Capacity | | | | | | | | | 2,700 | | | | | | |
| Activated Sludge - Additional Second. Clarifier Capacity | | | | 2,500 | | | | | 1,100 | | | | | | |
| UV Disinfection[1] | | | | 1,340 | | | | | | | | | | | |

| Table 11-1 WWTP Capital Improvement Schedule 2000-2020 ($1,000) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Improvement | 2001 | 2002 | 2003 | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | >2015 |
| Effluent Pump Station | | | | 400 | | | | | | | | | | | |
| Outfall | | | 1,200 | | | | | | | | | | | | |
| Sodium Hypochlorite | | | | 100 | | | | | | | | | | | |
| DAF Thickener | | | | | | | | | 400 | | | | | | |
| Additional Anaerobic Digester Capacity | | | | | | | | | 2,500 | | | | | | |
| Odor Control | | | | | | | | | | | | | 1,300 | | |
| Administration Building | | | | 600 | | | | | | | | | | | |
| Laborator/Operations Center | | | | | | | | | | | | | 600 | | |
| Shop/Garage | | | | 500 | | | | | | | | | | | |
| Flood Protection 100-year flood | | | | | | | | | | | | | | | 600 |
| Roadways | | | | | | | | | | | | | | | 250 |

| Table 11-1 WWTP Capital Improvement Schedule 2000-2020 ($1,000) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Improvement | 2001 | 2002 | 2003 | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | >2015 |
| Drainage Improvements | | | | | | | | | | | | | | | 50 |
| Total | 0 | 350 | 1,200 | 11,940 | 0 | 0 | 0 | 0 | 7,000 | 0 | 0 | 0 | 1,900 | 0 | 900 |

1. ENR Construction Cost Index 6397, October 2001.
2. Costs for UV disinfection include capital costs and pilot testing costs.

| Table 11-2 CSO Treatment Improvement Schedule 2000-2020 ($1,000) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Improvement | 2001 | 2002 | 2003 | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | >2015 |
| CSO Treatment - Park Street Conveyance | | | | | | | | | | | | | 1,900 | | |
| CSO Treatment - High Rate Clarification | | | | | | | | | | | | | 4,200 | | |
| CSO Treatment - UV Disinfection | | | | | | | | | | | | | 2,200 | | |
| CSO Treatment - Effluent Pump Station | | | | | | | | | | | | | 800 | | |
| Total | | | | | | | | | | | | | 9,100 | | |

1. ENR Construction Cost Index 6397, October 2001.
3. Costs as presented in Chapter 4, Combined Sewer System

| Table 11-3 Collection System Improvement Schedule 2000-2020 ($1,000) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Improvement | 2001 | 2002 | 2003 | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | >2015 |
| FS-1 Sections 23 and 26 | | | | | | | | | | | | | | | 380 |
| FS-2 Sections 15 and 22 | | | | | | | | | | | | | | | 295 |
| FS-3 Martin Road | | | | | | | | | | | | | | | 135 |
| FS-4 College Way | | | | | | | | | | | | | | | 125 |
| FS-5 College Way | | 635 | | | | | | | | | | | | | |
| FS-6 Fir Street | | | | | 270 | | | | | | | | | | |
| FS-7 Fir Street | | | | | 350 | | | | | | | | | | |
| FS-8 26th Street | | | | | | | | | | | | | | | 190 |
| FS-9 26th Street | | | | | | | | | | | | | | | 140 |
| FS-10 LaVenture Rd | | | | | | | | | | | | | | | 235 |
| FS-11 LaVenture Rd | | | | | | | | | | | | | | | 75 |
| FS-12 LaVenture Rd | | | | | | | | | | | | | | | 255 |

| Table 11-3 Collection System Improvement Schedule 2000-2020 ($1,000) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Improvement | 2001 | 2002 | 2003 | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | >2015 |
| FS-13 Alder Ln Inter. | | | | | | | | | | | | | | | 220 |
| FS-14 Urban Ave | | | | | | | | | | | | | | | 70 |
| FS-15 Freeway Dr | | | | | | | | | | | | | | | 240 |
| FS-16 West Mount Vernon | | | | | | | | | | | | | | | 150 |
| FS-17 Central CSO Regulator | 30 | | | | | | | | | | | | | | |
| CS-1 Snoqualmie | 20 | | | | | | | | | | | | | | |
| CS-2 1115 N. 8th | 20 | | | | | | | | | | | | | | |
| CS-3 S. 7th | 20 | | | | | | | | | | | | | | |
| CS-4 N 6th | 20 | | | | | | | | | | | | | | |
| CS-5 Brick Hill | 30 | | | | | | | | | | | | | | |
| CS-6 Blodgett Rd | 20 | | | | | | | | | | | | | | |
| CS-7 Kincaid | 20 | | | | | | | | | | | | | | |
| CS-8 S 20th | 20 | | | | | | | | | | | | | | |
| CS-9 Section | 50 | | | | | | | | | | | | | | |

| Table 11-3 Collection System Improvement Schedule 2000-2020 ($1,000) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Improvement | 2001 | 2002 | 2003 | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | >2015 |
| CS-10 Douglas/ Walter Alley | 75 | | | | | | | | | | | | | | |
| CS-11 107 Cedar | 45 | | | | | | | | | | | | | | |
| CS-12 N 6th | 60 | | | | | | | | | | | | | | |
| CS-13 Section | 5 | | | | | | | | | | | | | | |
| CS-14 Broadway | 20 | | | | | | | | | | | | | | |
| CS-15 Broad St | 20 | | | | | | | | | | | | | | |
| CS-16 and CS-31 Interstate 5[4] | | | | 750 | | | | | | | | | | | |
| CS-17 Division Alley | 5 | | | | | | | | | | | | | | |
| CS-18 Bernice | 5 | | | | | | | | | | | | | | |
| CS-20 Lawrence | 5 | | | | | | | | | | | | | | |
| CS-21 1224 12th S | 25 | | | | | | | | | | | | | | |
| CS-22, CS-23 and CS-29 8th St Improvements | | | 1,000 | | | | | | | | | | | | |
| CS-25 Carpenter Alley | 5 | | | | | | | | | | | | | | |

| Table 11-3 Collection System Improvement Schedule 2000-2020 ($1,000) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Improvement | 2001 | 2002 | 2003 | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | >2015 |
| CS-26 1120 N 16th | 5 | | | | | | | | | | | | | | |
| CS-27 1210 N 14th | 5 | | | | | | | | | | | | | | |
| CS-28 8th | 5 | | | | | | | | | | | | | | |
| CS-32 N 1st | 5 | | | | | | | | | | | | | | |
| CS-34 Christenson Seed West | 5 | | | | | | | | | | | | | | |
| CS-35 Cleveland | 20 | | | | | | | | | | | | | | |
| CS-40 Lind St | 5 | | | | | | | | | | | | | | |
| Total | 570 | 635 | 1,000 | 750 | 620 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2,510 |

1. ENR Construction Cost Index 6397, October 2001.
2. Costs for the I-5 improvements have been estimated at $750,000 for all crossings. Actual cost estimates will vary depending upon the required improvements after all the crossing have been evaluated. See Chapter 5 for additional details..

| Table 11-4 Summary of Capital Improvement Schedule 2000-2020 ($1,000) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Improvement** | **2001** | **2002** | **2003** | **2004** | **2005** | **2006** | **2007** | **2008** | **2009** | **2010** | **2011** | **2012** | **2013** | **2014** | **>2015** |
| Wastewater Treatment Facility | 0 | 350 | 1,200 | 11,940 | 0 | 0 | 0 | 0 | 7,000 | 0 | 0 | 0 | 1,900 | 0 | 900 |
| CSO Treatment | | | | | | | | | | | | | 9,100 | | |
| Collection System | 570 | 635 | 1,000 | 750 | 620 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2,510 |
| **Total** | 570 | 985 | 2,200 | 12,690 | 620 | 0 | 0 | 0 | 7,000 | 0 | 0 | 0 | 11,000 | 0 | 3,410 |

1. ENR Construction Cost Index 6397, October 2001.

**Copies of Figures No. 3-1, 5-1, 5-2, and 5-3 can be viewed at the CEDD Department. Copies were not made of these maps because they are oversized maps.**