City of Mount Vernon
Comprehensive Sewer Plan Update
February 2003
Final

Prepared for:

City of Mount Vernon

Prepared by:

HDR Engineering, Inc.

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:

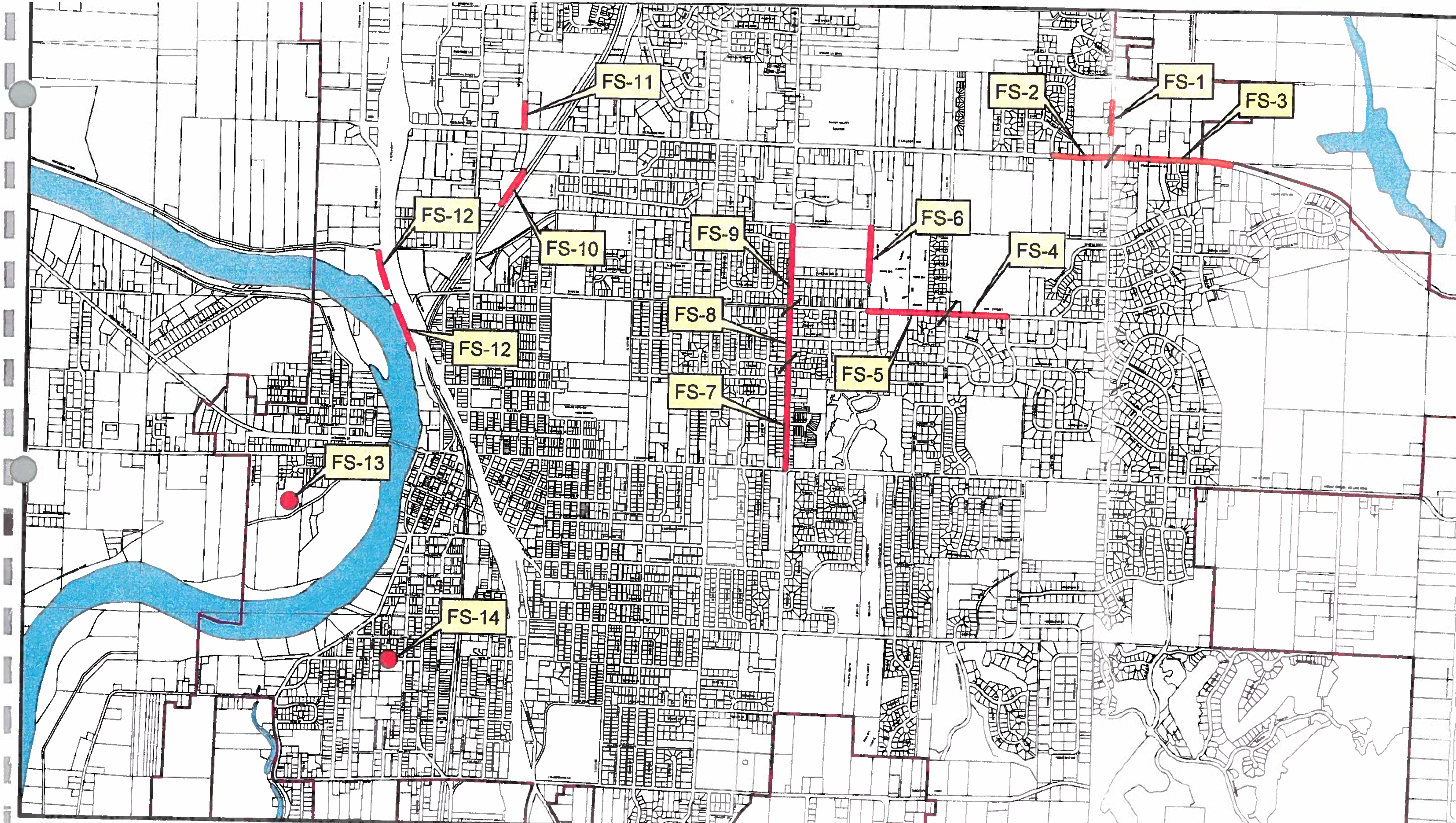
- Convert the Aeration Basin No. 4 to an Activated Sludge Aeration Basin,
- Construct additional Aeration Basins,
- Convert Secondary Clarifier No. 1 to an Aerobic Digester,
- Construct an additional Secondary Clarifier,
- Construct an additional Dissolved Air Floatation Thickener,
- Construct an additional 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

Mount Vernon Comprehensive Sewer Plan	Date: Dec 2002
Interceptor Improvements	Figure No. 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 NO. 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	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	--

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.

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ABBREVIATIONS

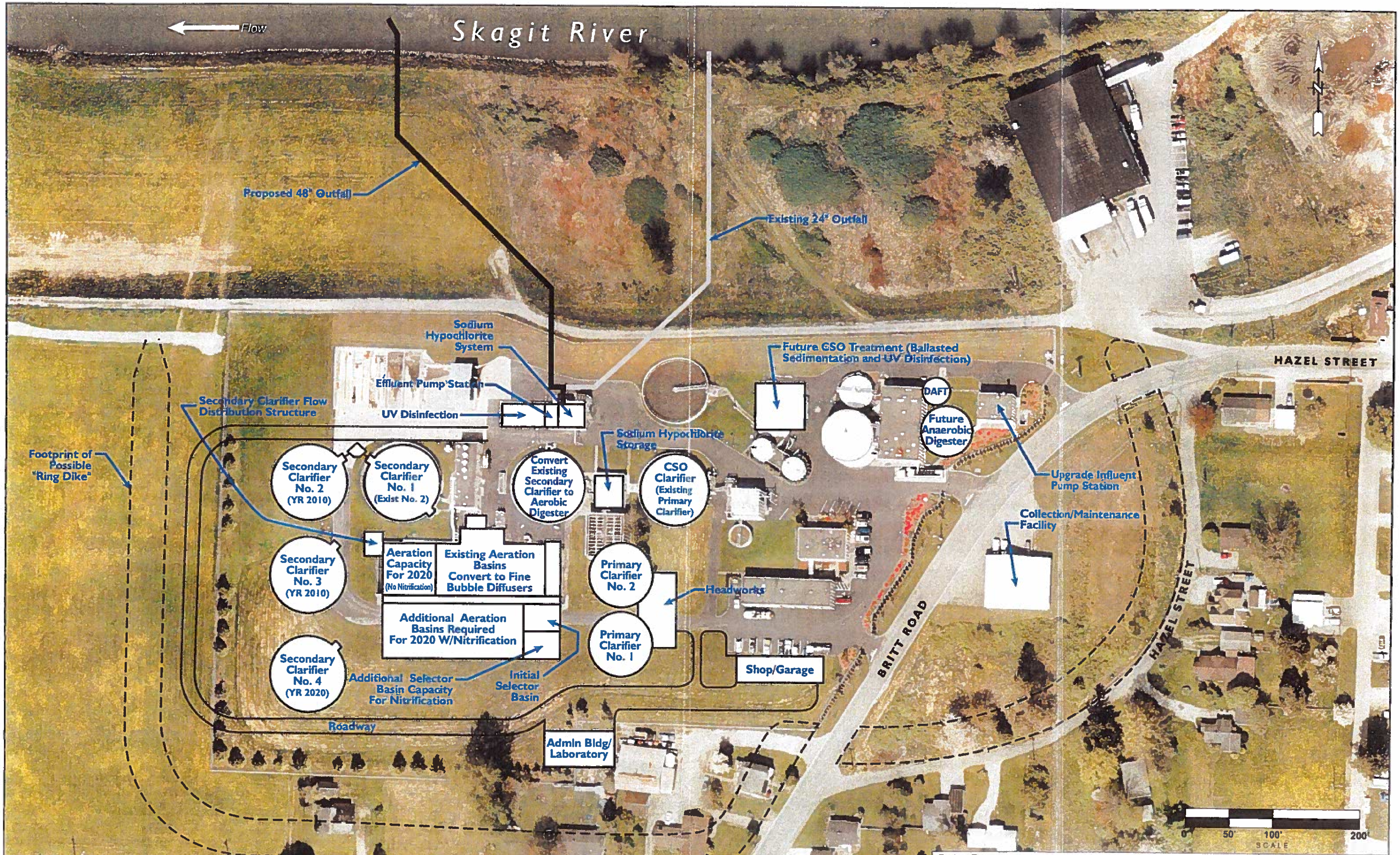
ac	Acre
ADMM	Average Day Maximum Month
AKART	All known, available, and reasonable methods of treatment
BAT	Best Available Technology Economically Achievable
BE/BA	Biological Evaluation/Biological Assessment
BOD	Biochemical Oxygen Demand
BPT	Best Practical Control Technology Currently Available
$CBOD_5$	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



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 to 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.

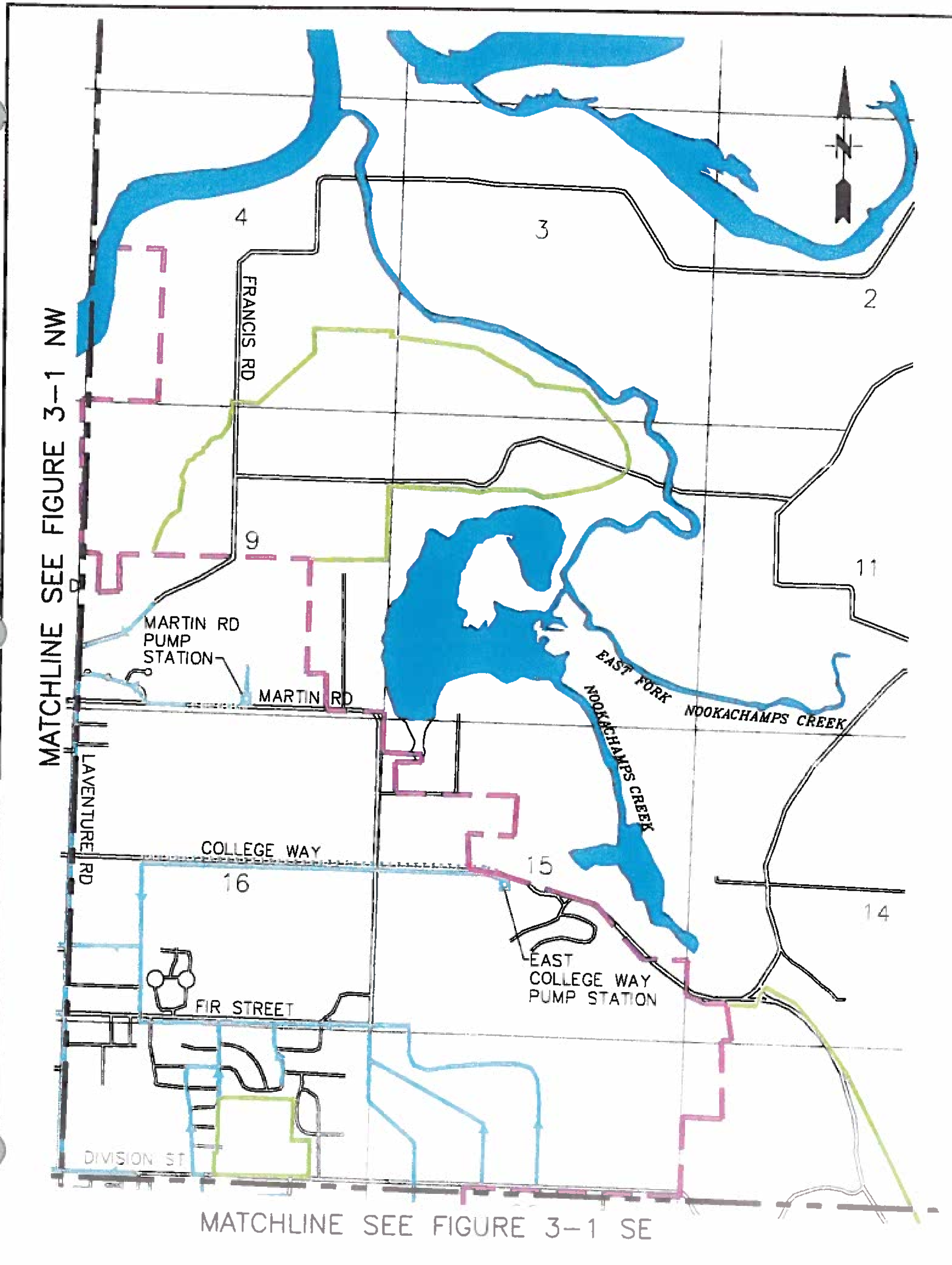
Existing Sewer Service Area

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.



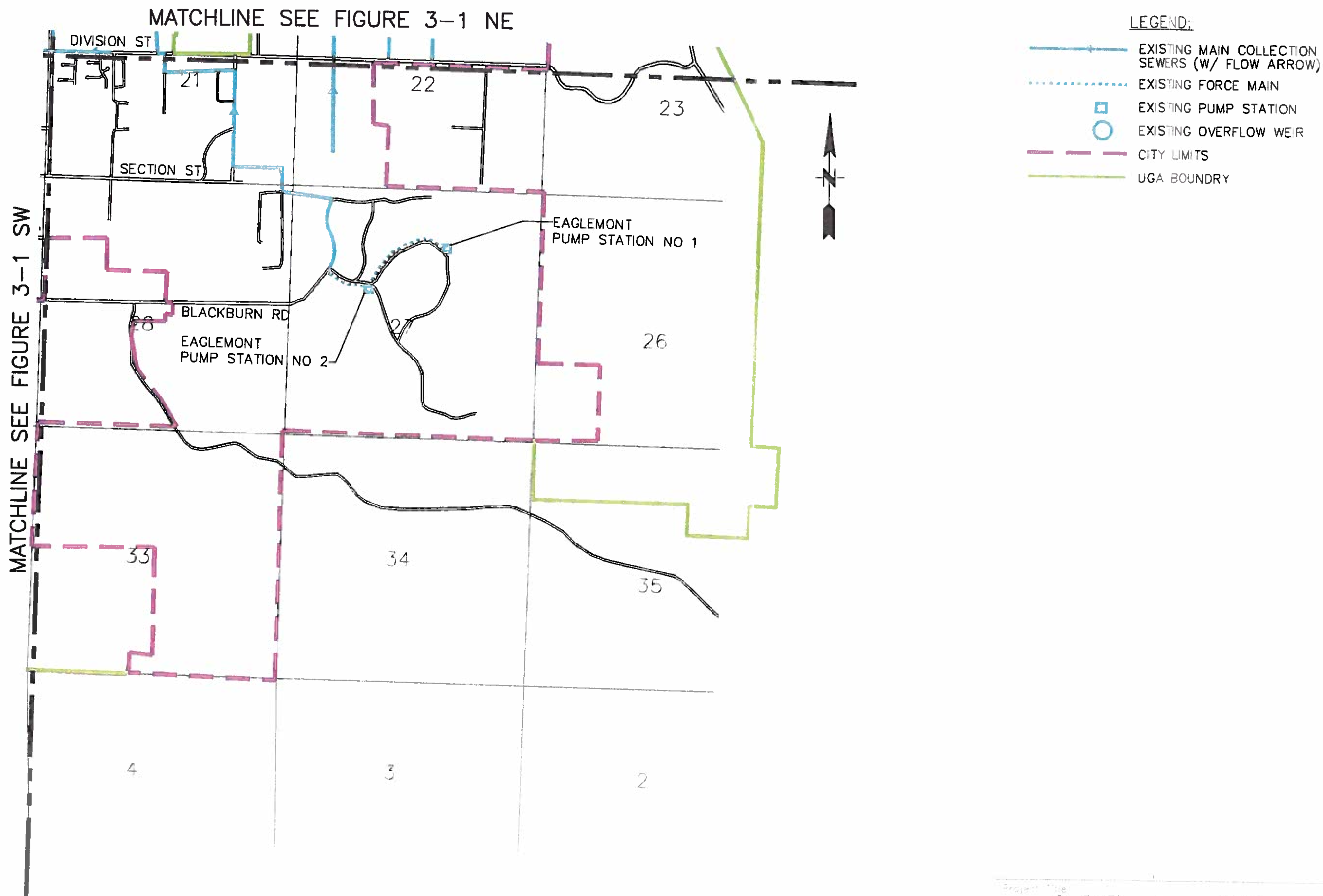
HDR Engineering, Inc

Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No: 3-1 NE



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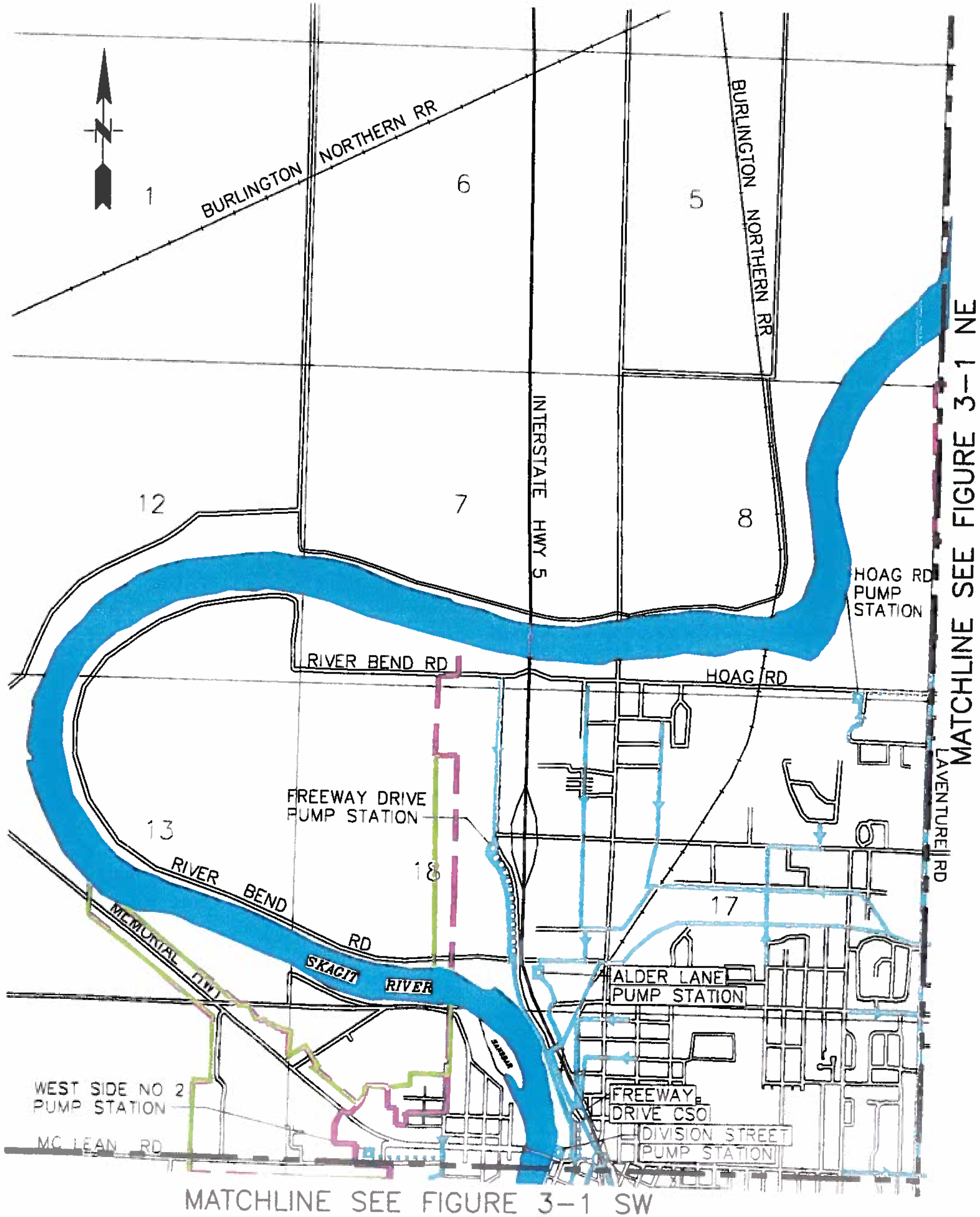
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

FEBRUARY 2003

EXISTING COLLECTION SYSTEM

3–1 SE

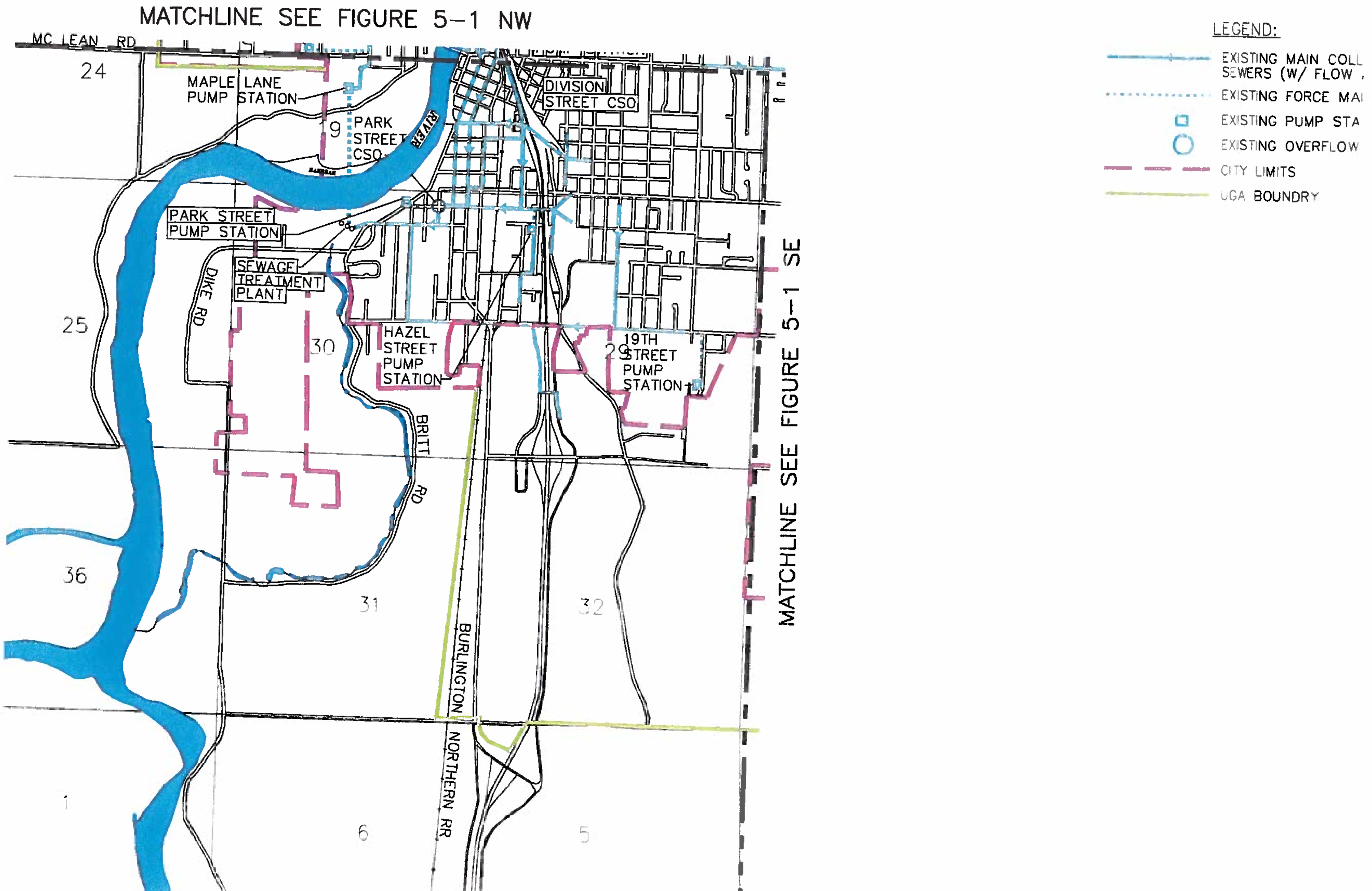


LEGEND:

- 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	Date
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE	FEBRUARY 2003
Sheet Title	Figure No.
EXISTING COLLECTION SYSTEM	3-1 NW



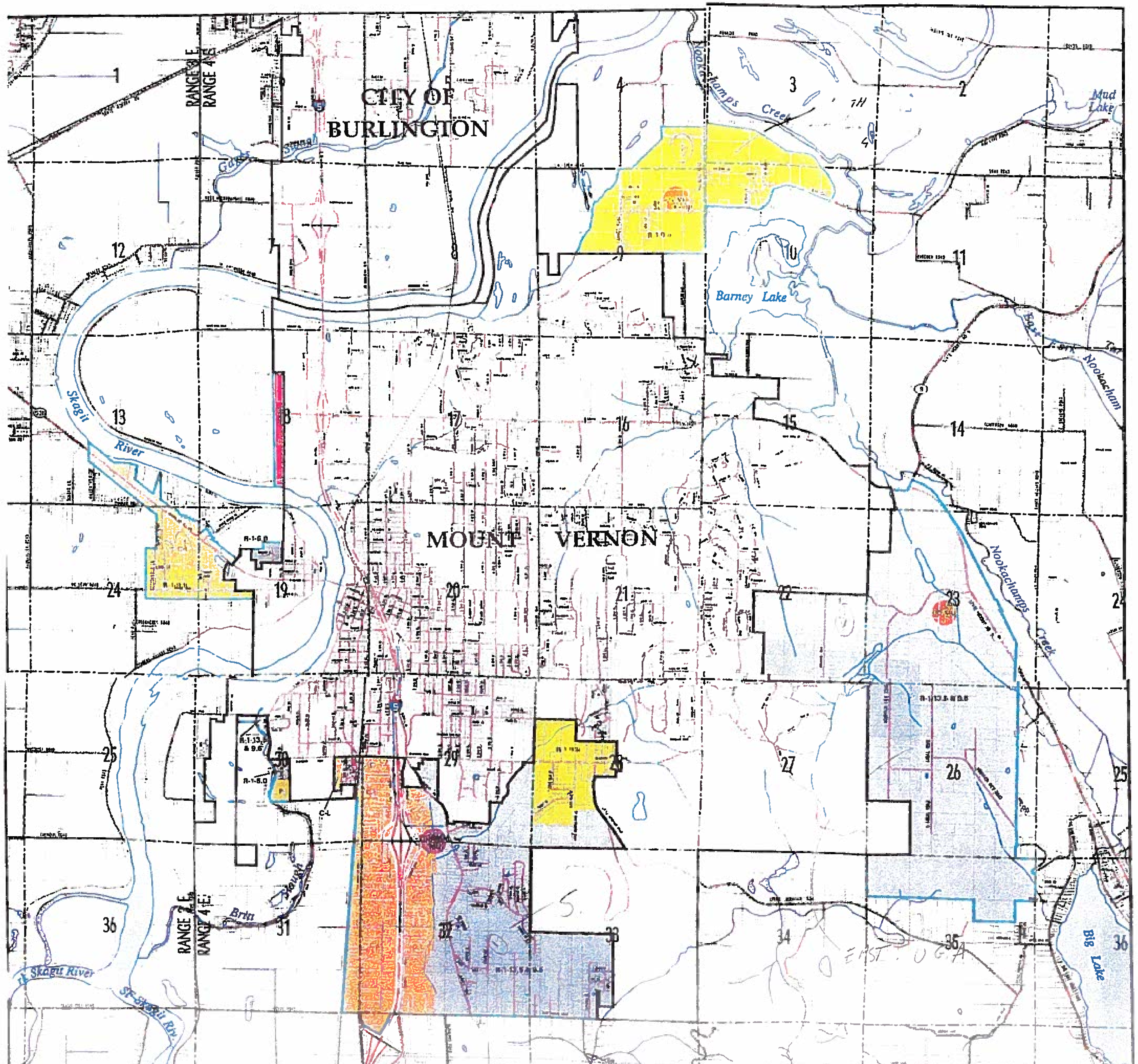
HDR Engineering, Inc

MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

FEBRUARY 2003

EXISTING COLLECTION SYSTEM

3-1 SW



LEGEND

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

* 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 16, 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.

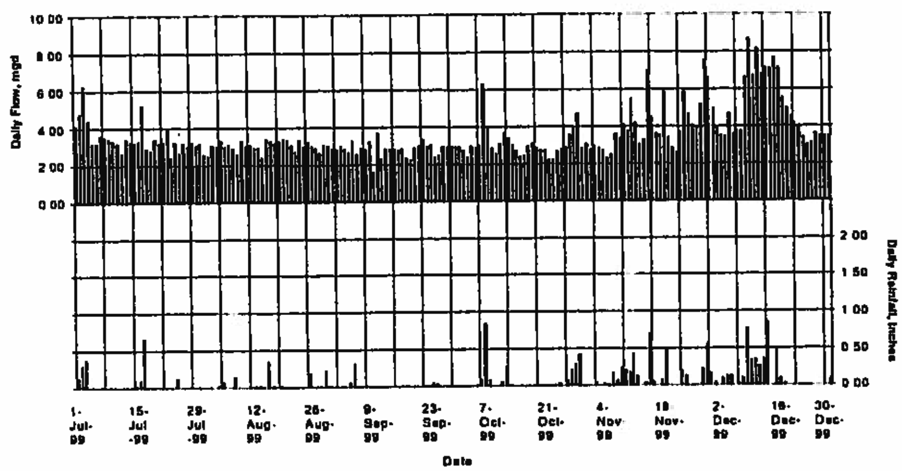
HISTORICAL FLOWS AND LOADS

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

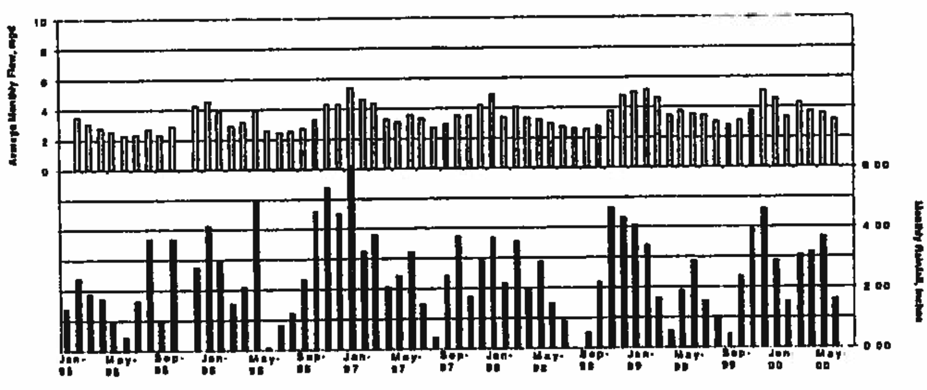
December 31, 1999. This plot illustrates that during late summer the flows reach a base rate of about 2.6 mgd. The plot also illustrates in the dry weather period the five day work week of Draper Valley Farms(DVF), Inc, which discharges from 0.4 to 0.6 mgd when in operation. In November and December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

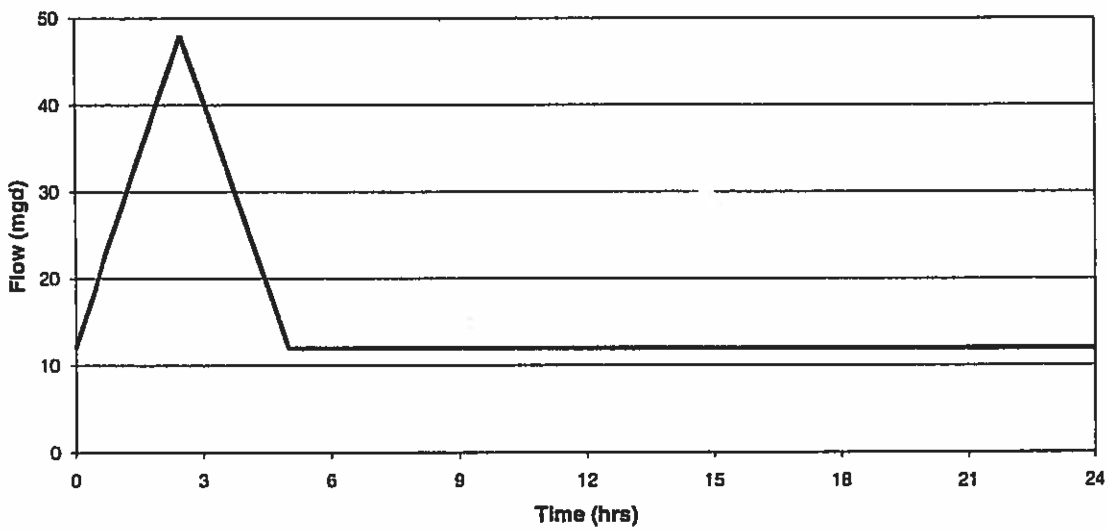
The CSO Baseline for Mount Vernon was established in 1988. It predicted an annual CSO volume of 116.5 MG for the average annual rainfall of 31.95 inches. Based on the 1988 collection system there was a 95 percent confidence that the volume, with an average annual rainfall, would be between 92 MG and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

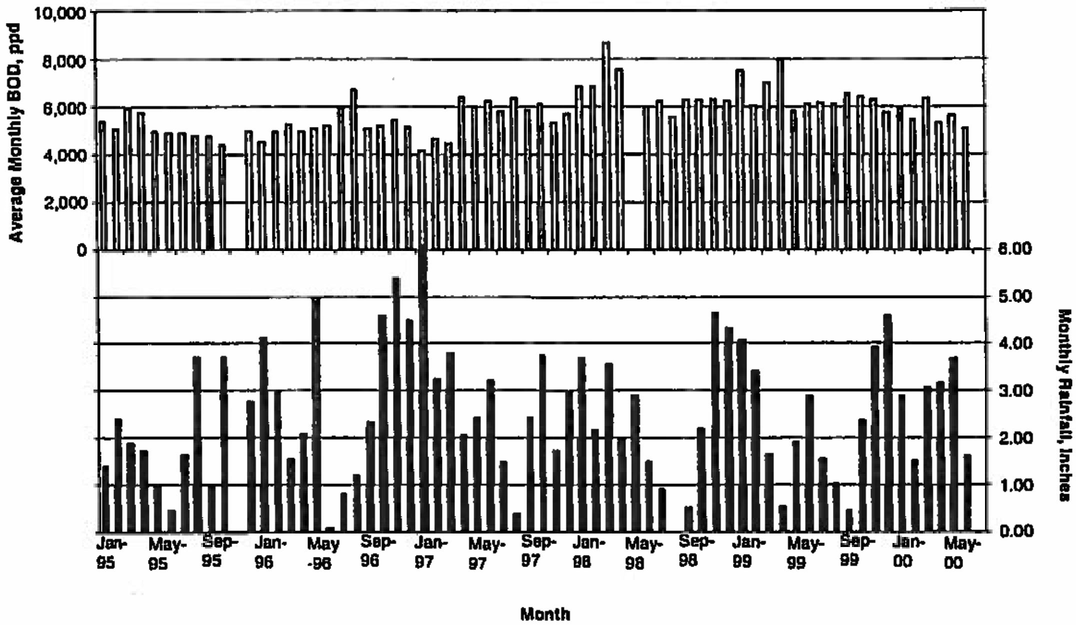
1. Storm flow component estimated from May 16, 1988, storm event.
2. Sanitary and storm components combined flow estimates

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.

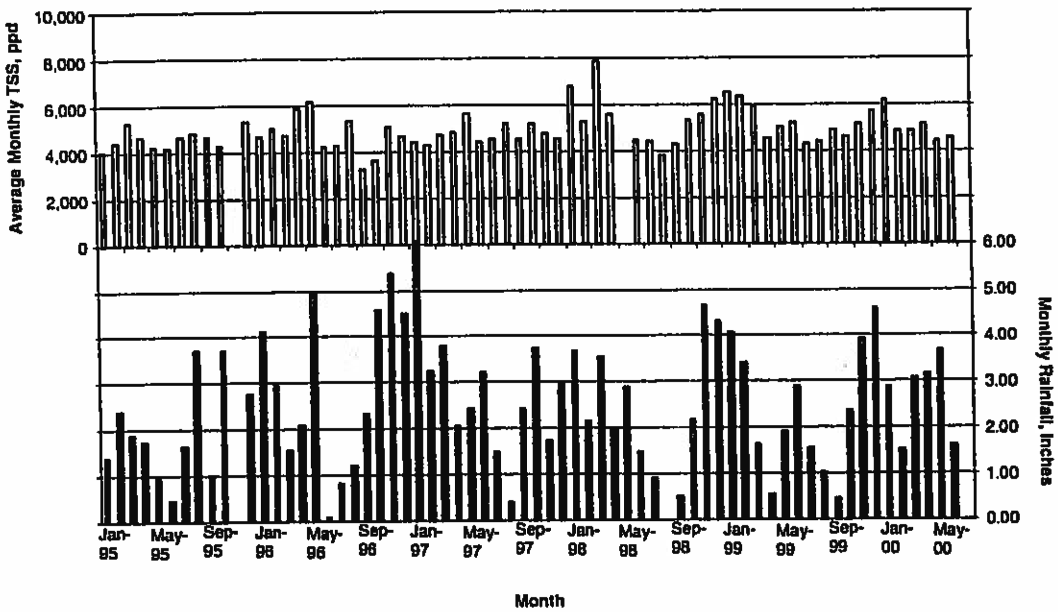
The only current industrial customer, Draper Valley Farms, Inc. (DVF), discharges between 500 and 1,200 ppd of BOD per month. Additional information on these loads Is provided in Chapter 6. Based on previous City discussions with DVF it is assumed that the future flows from the plant could approach 0.75 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

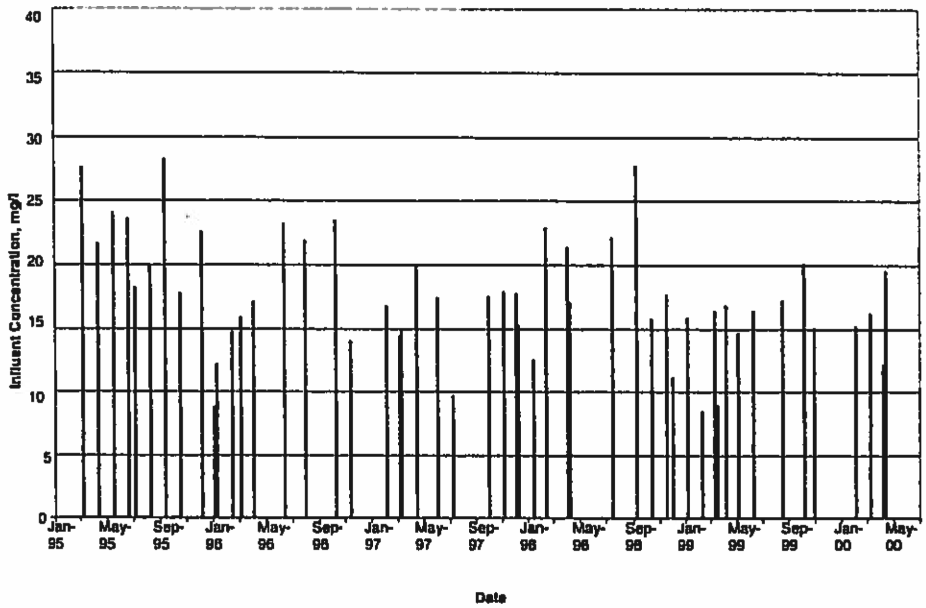
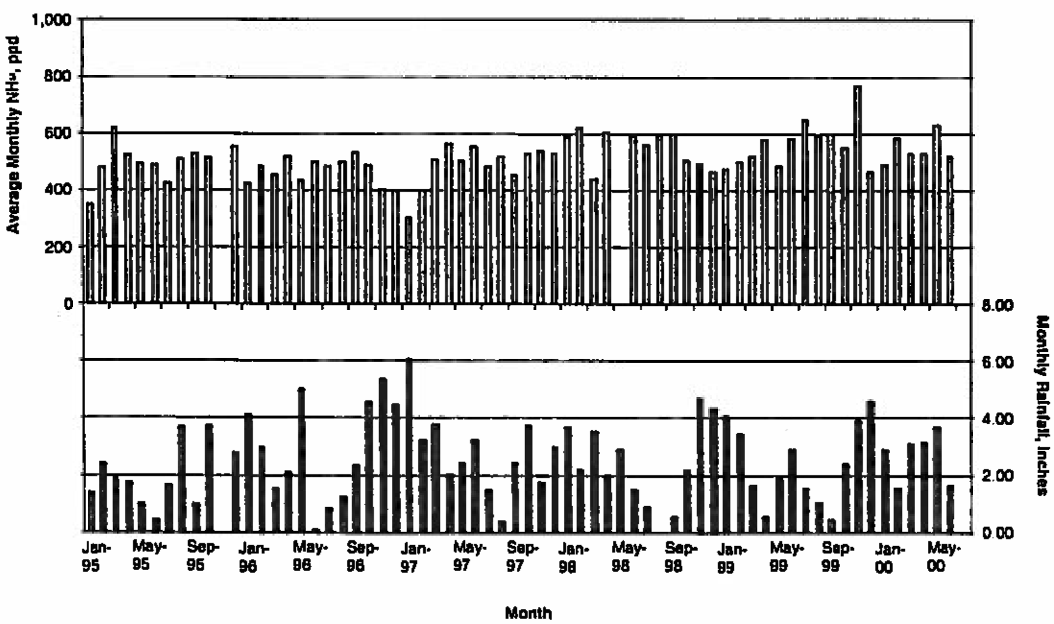


Figure 3-9 City of Mount Vernon Monthly Ammonia Loading



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 mgd[2]	18.3 mgd[3]

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

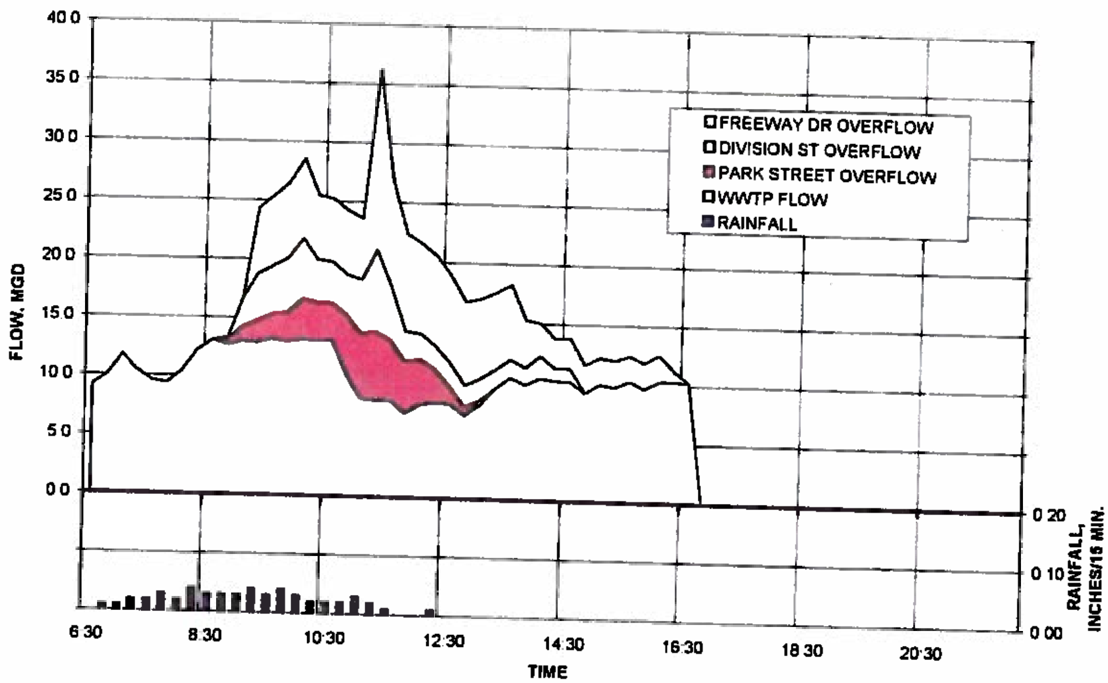
Combined Sewer Overflows from November 1998 to 2000

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



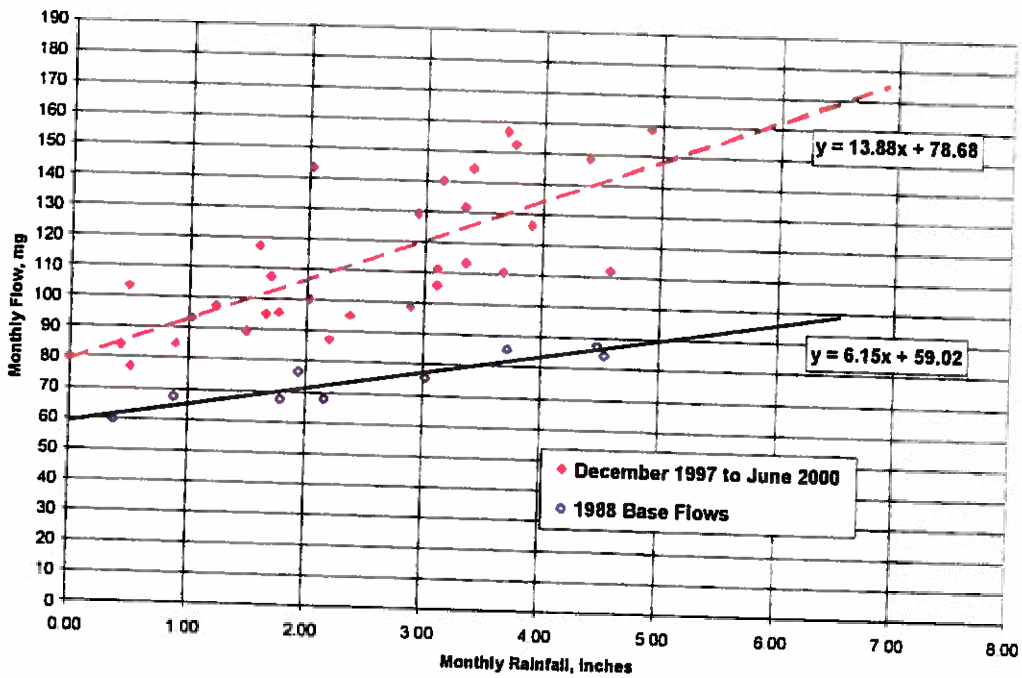
The October 1995 Wastewater Treatment Plant Evaluation evaluated the facilities required based on a peak design system flow rate of 48 mgd. The peak system flow rate observed since the Central CSO Regulator has been in service was 36.2 mgd. A detailed evaluation of the return frequency of this flow rate 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

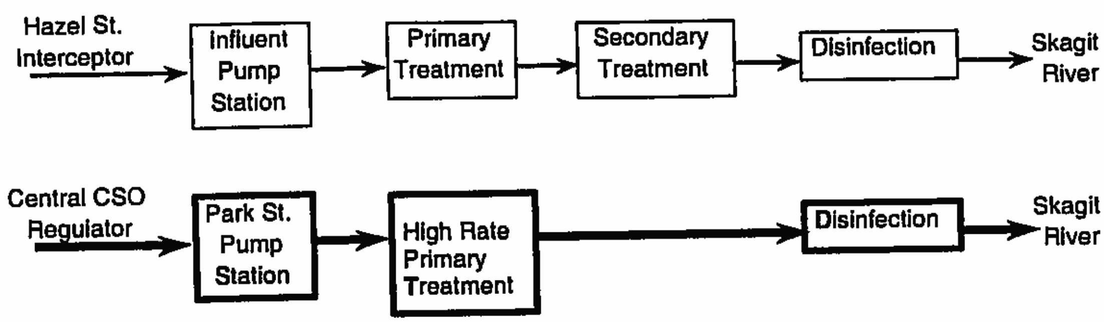
The State of Washington defines CSO treatment as primary treatment that removes at least 50 percent of the total suspended solids (TSS) and an average settlable solids concentration of 0.3 mL/L/hr, with a maximum of 1.9 mL/L/hr. Based on recent CSO treatment projects, the Department of Ecology has interpreted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



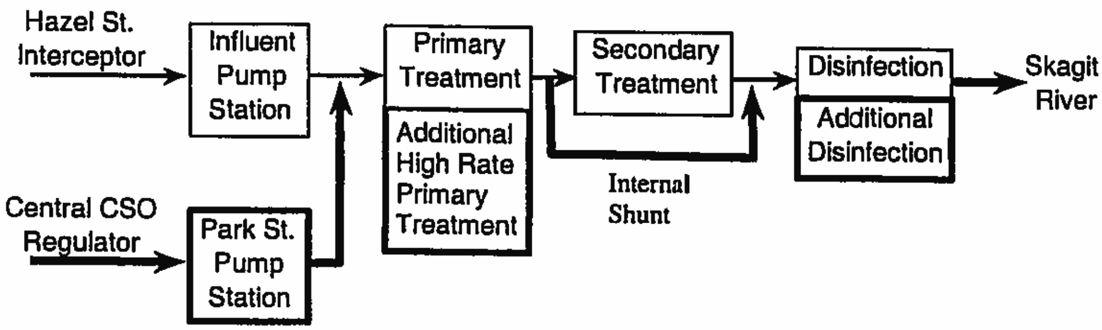
The improvements required for Alternative 1 include:

- 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



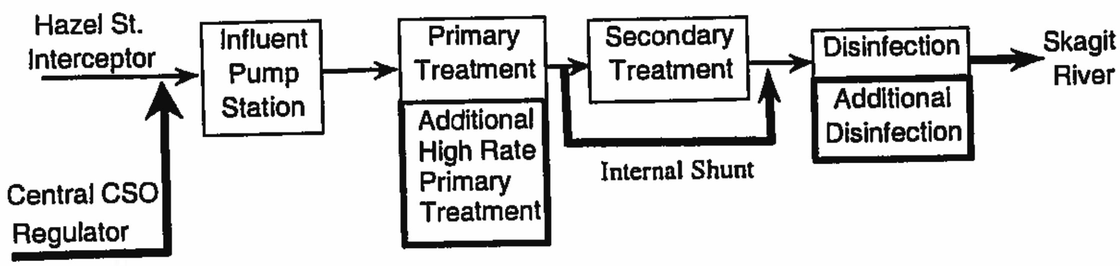
The improvements required for Alternative 2 include:

- 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: 30 mg/L BOD and TSS	NPDES Permit Limits: 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.

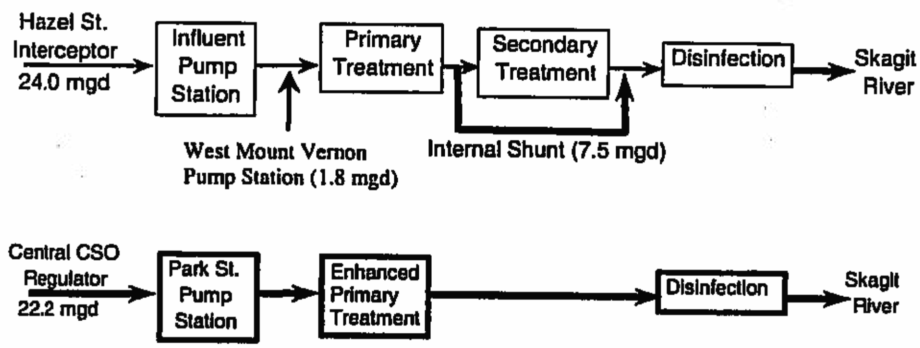
Treatment Recommendation

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. 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. 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. Construct a UV disinfection system Construct a 750 LF of 36-inch sewer. Upgrade Park Street Pump Station Construct 1500 LF of 30-inch force main Construct a CSO effluent pump station 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

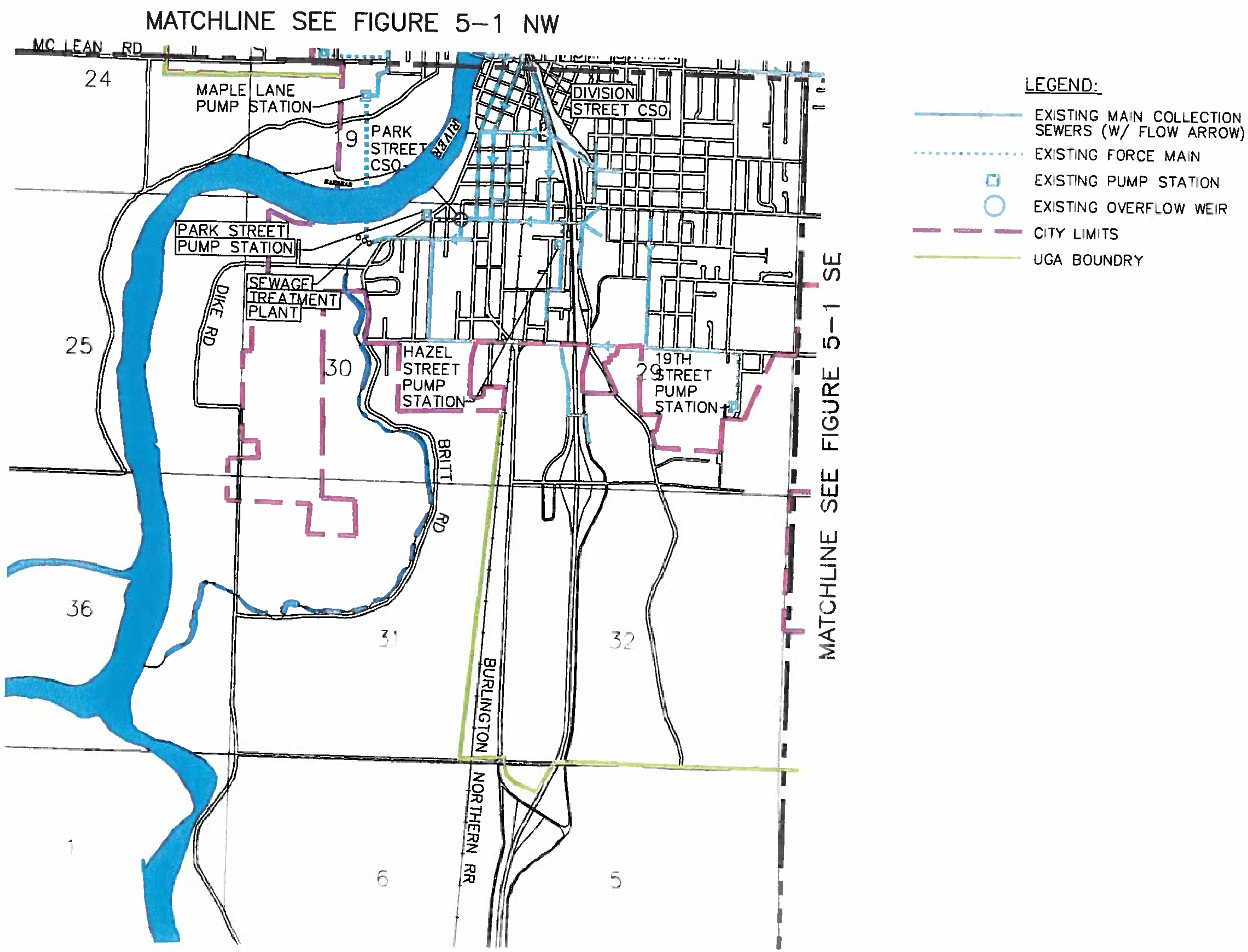
This chapter presents an evaluation of the wastewater collection system. It includes a review of the interceptor system capacity based on projected peak flows, using the population projections presented in Chapter 3. A review of the City's Access database of sewer defects was also completed. The following 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.



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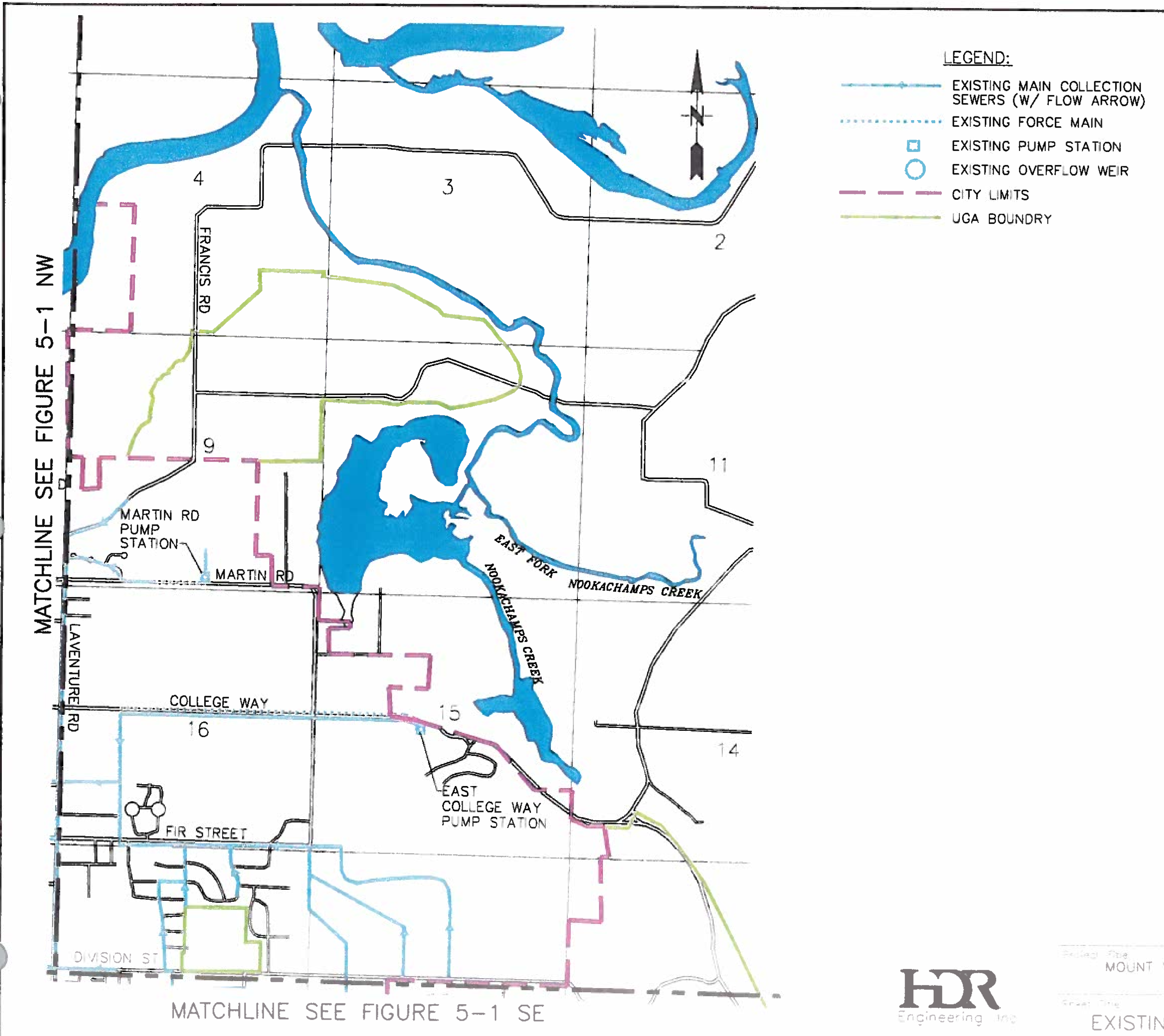
Date: FEBRUARY 2003

Sheet Title: EXISTING COLLECTION SYSTEM

Figure No. 5-1 SW



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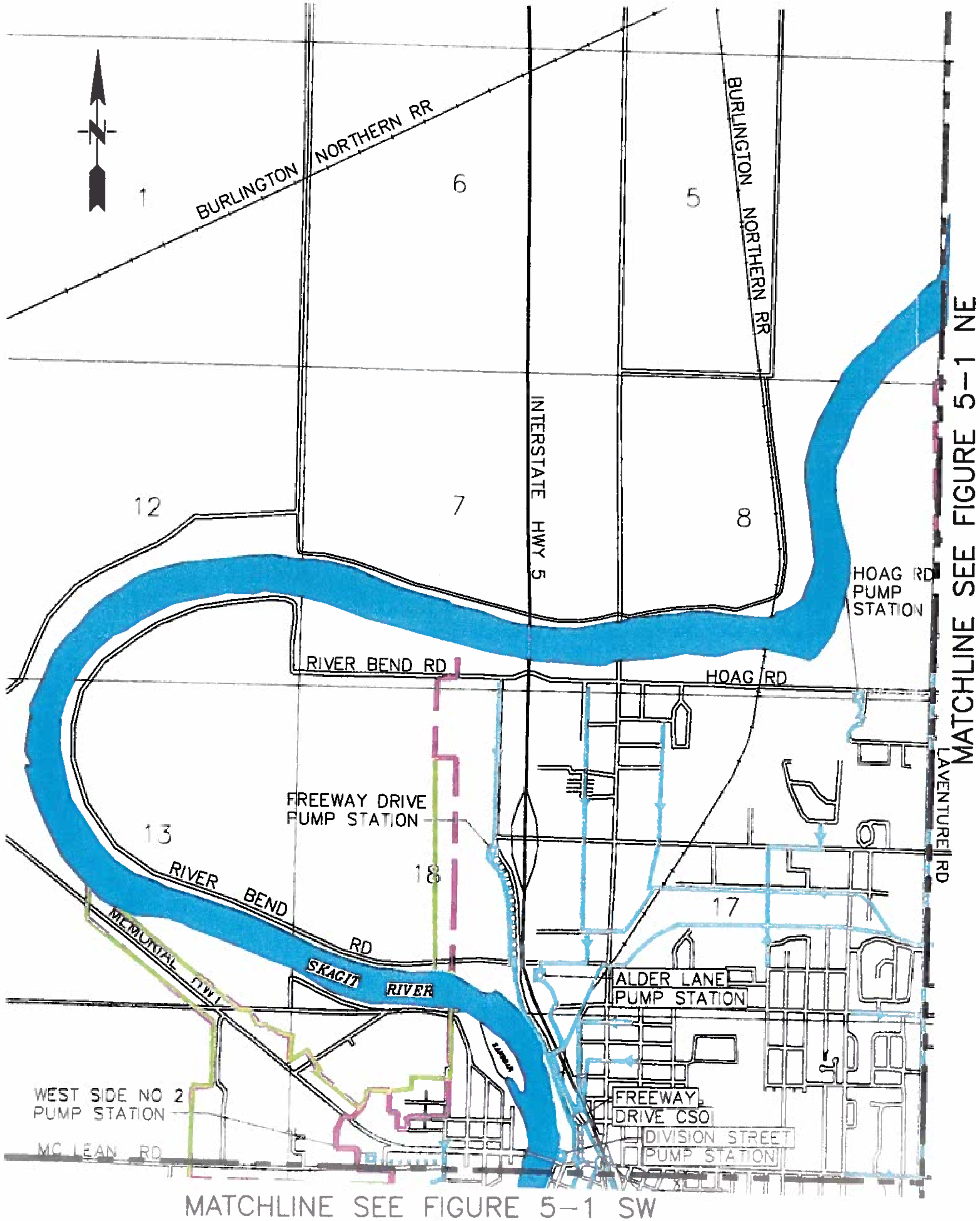
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

EXISTING COLLECTION SYSTEM

FEBRUARY 2003

5-1 NE



LEGEND:

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

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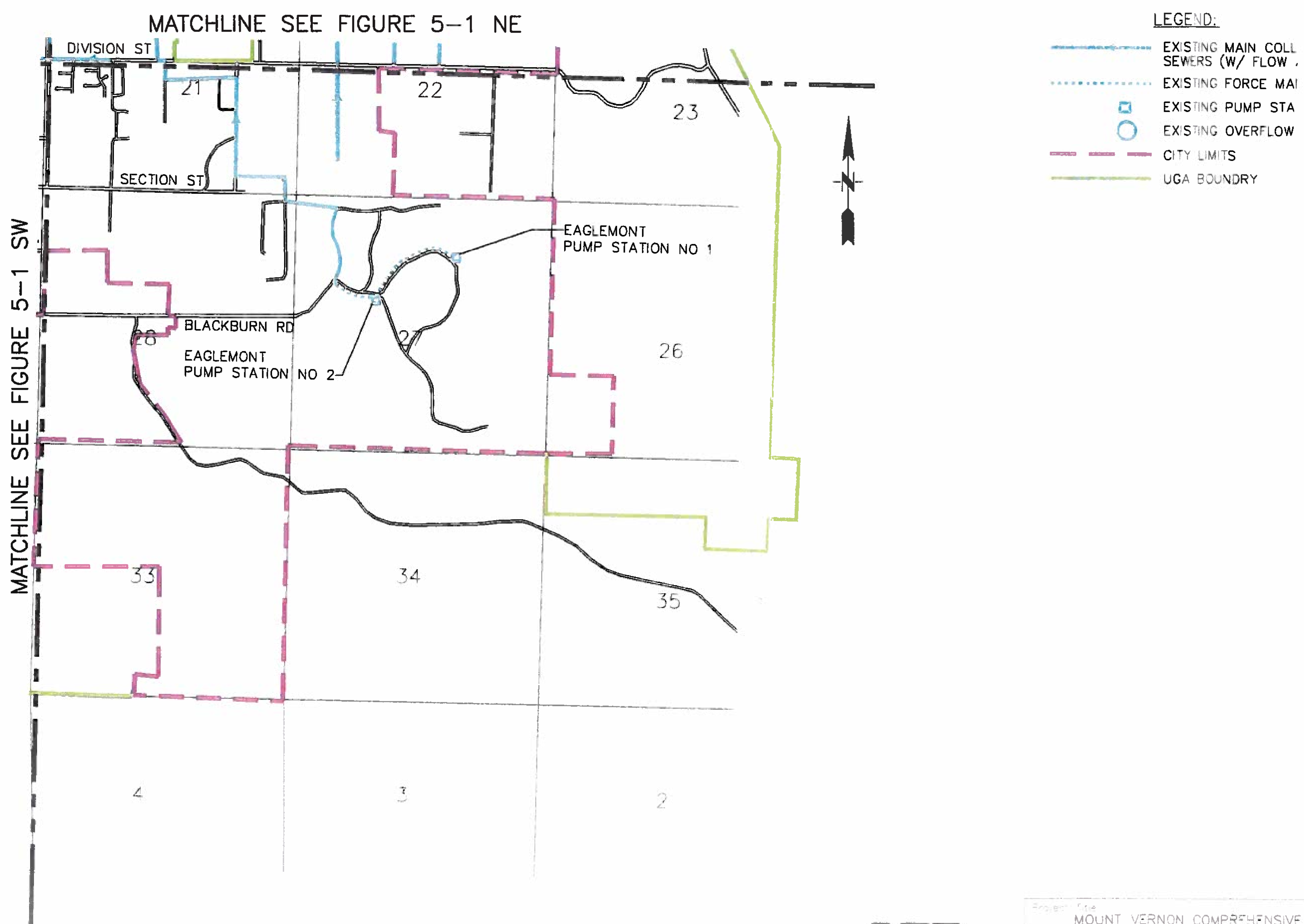
Project Title	MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE	Date FEBRUARY 2003
Sheet Title	EXISTING COLLECTION SYSTEM	Figure No. 5-1 NW



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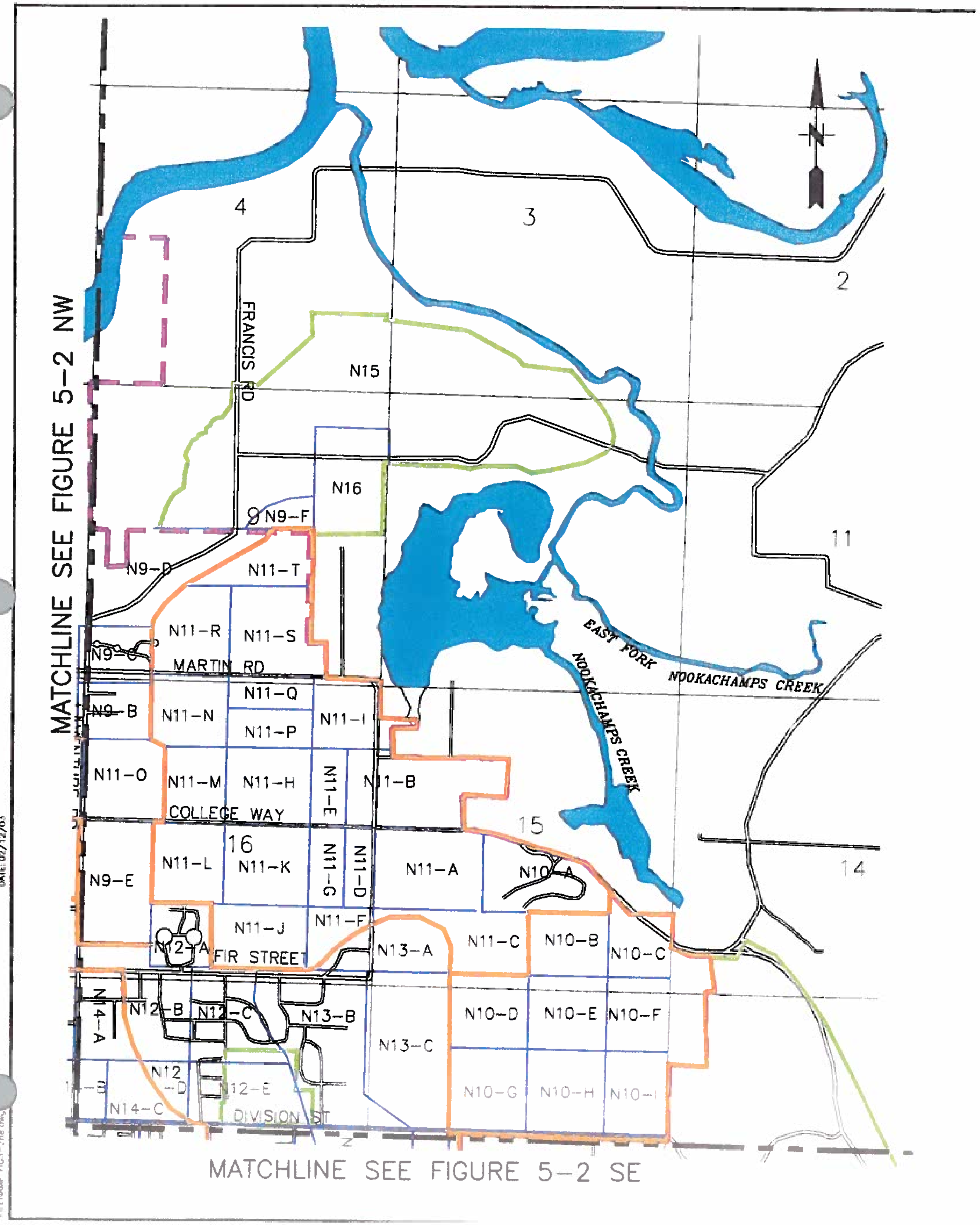
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

FEBRUARY 2003

EXISTING COLLECTION SYSTEM

5-1 SE



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

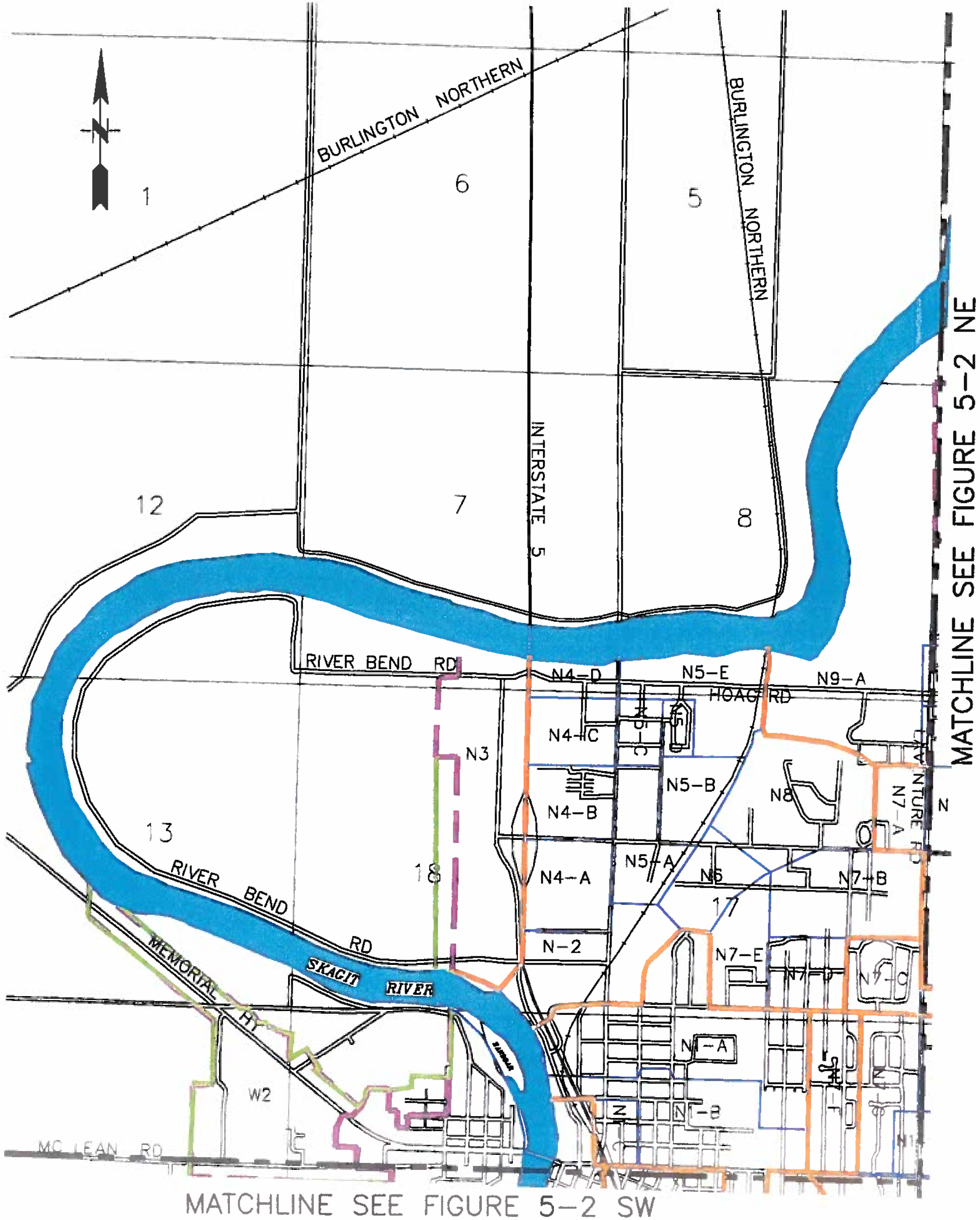
The system analysis was completed by defining the interceptors and major trunk lines. Manhole invert elevations and pipe lengths between manholes in the defined segments were obtained from City utility mapping and previous HYDRA modeling efforts. The analysis was completed by developing flow components 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

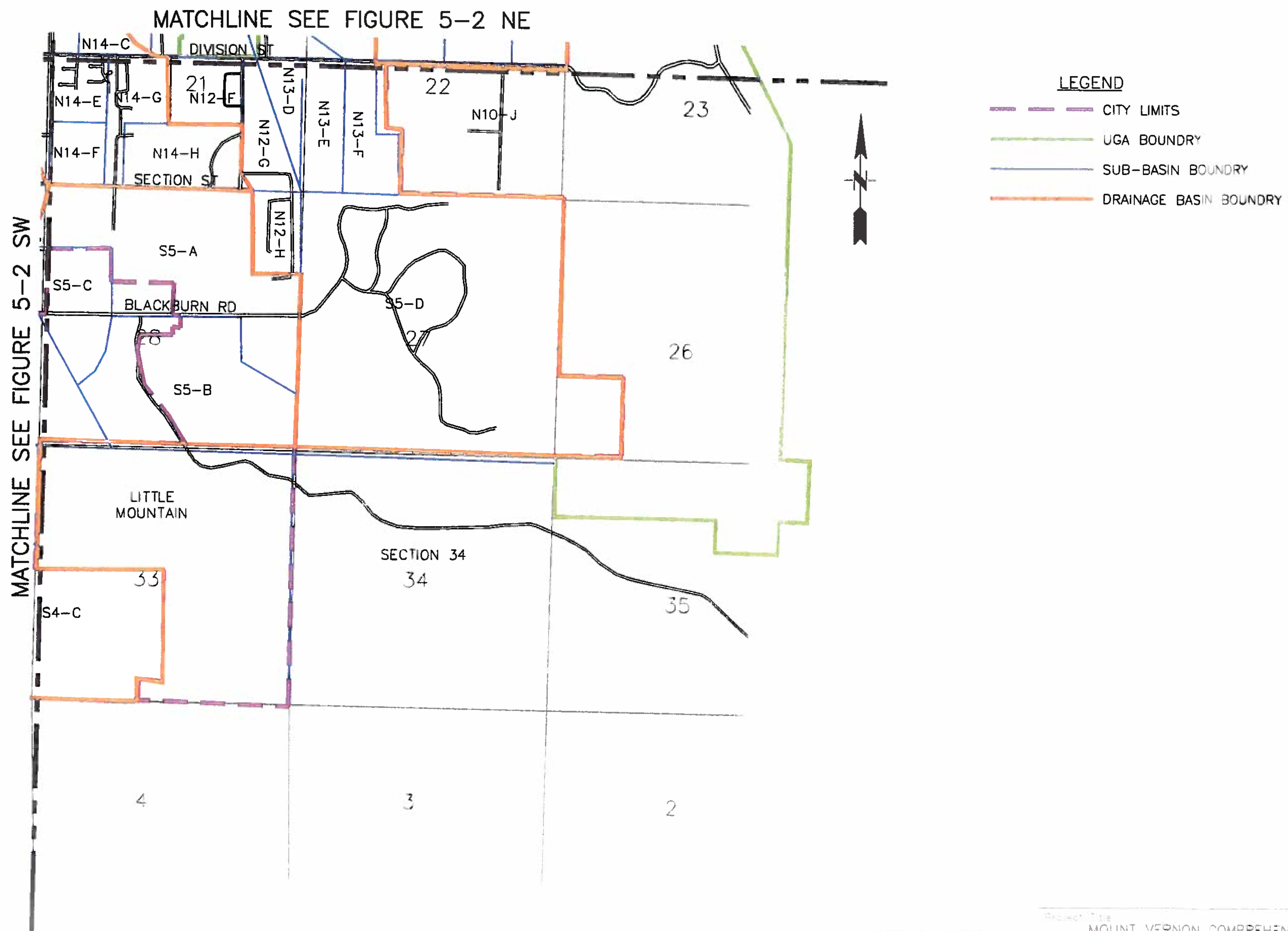
HDR Engineering, Inc

Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: DRAINAGE BASINS

Figure No. 5-2 NW



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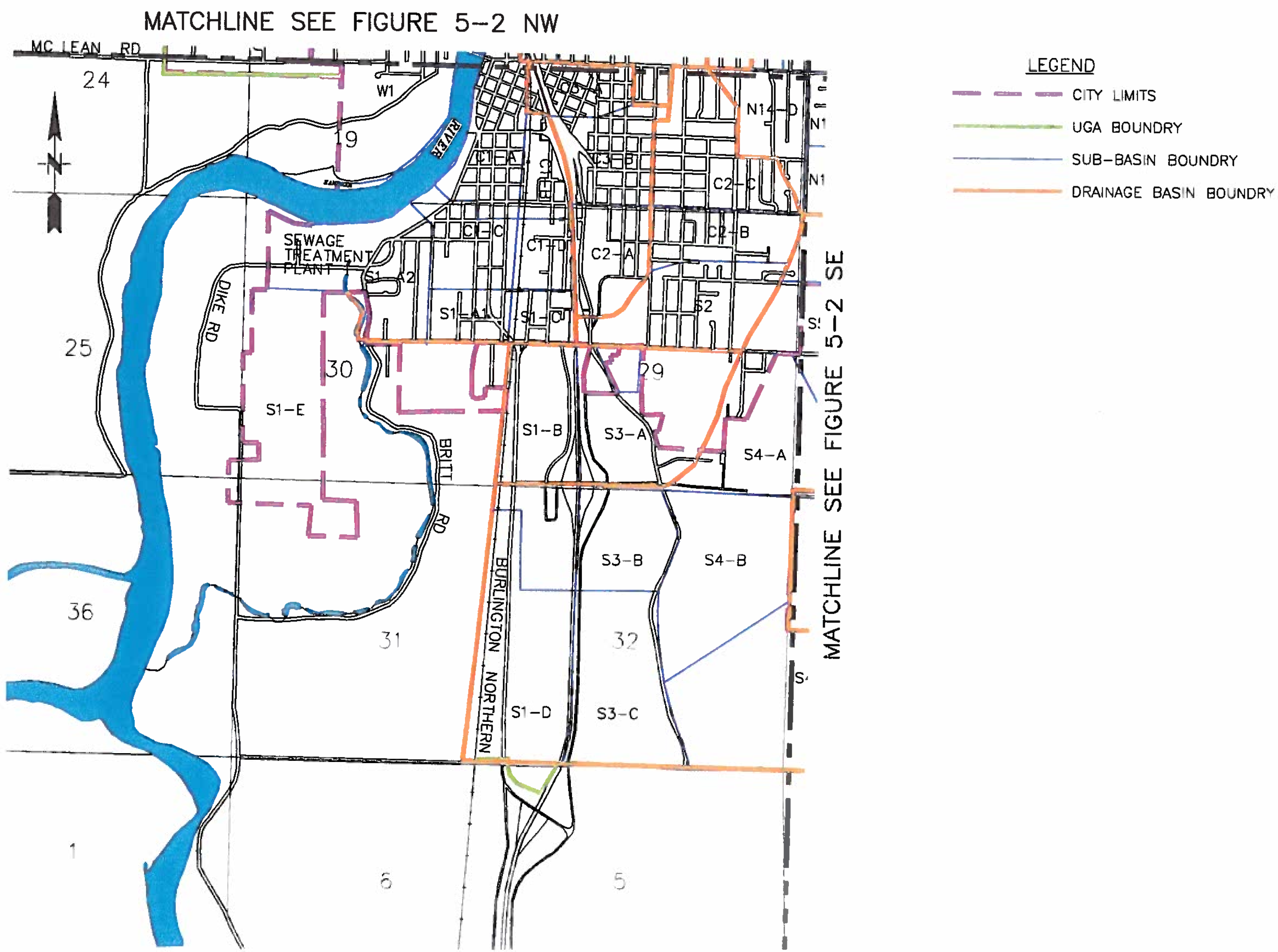
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

FEBRUARY 2003

DRAINAGE BASINS

5-2 SE

DATE: 02/12/03



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Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE	Date: FEBRUARY 2003
Sheet Title: DRAINAGE BASINS	Figure No. 5-2 SW

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

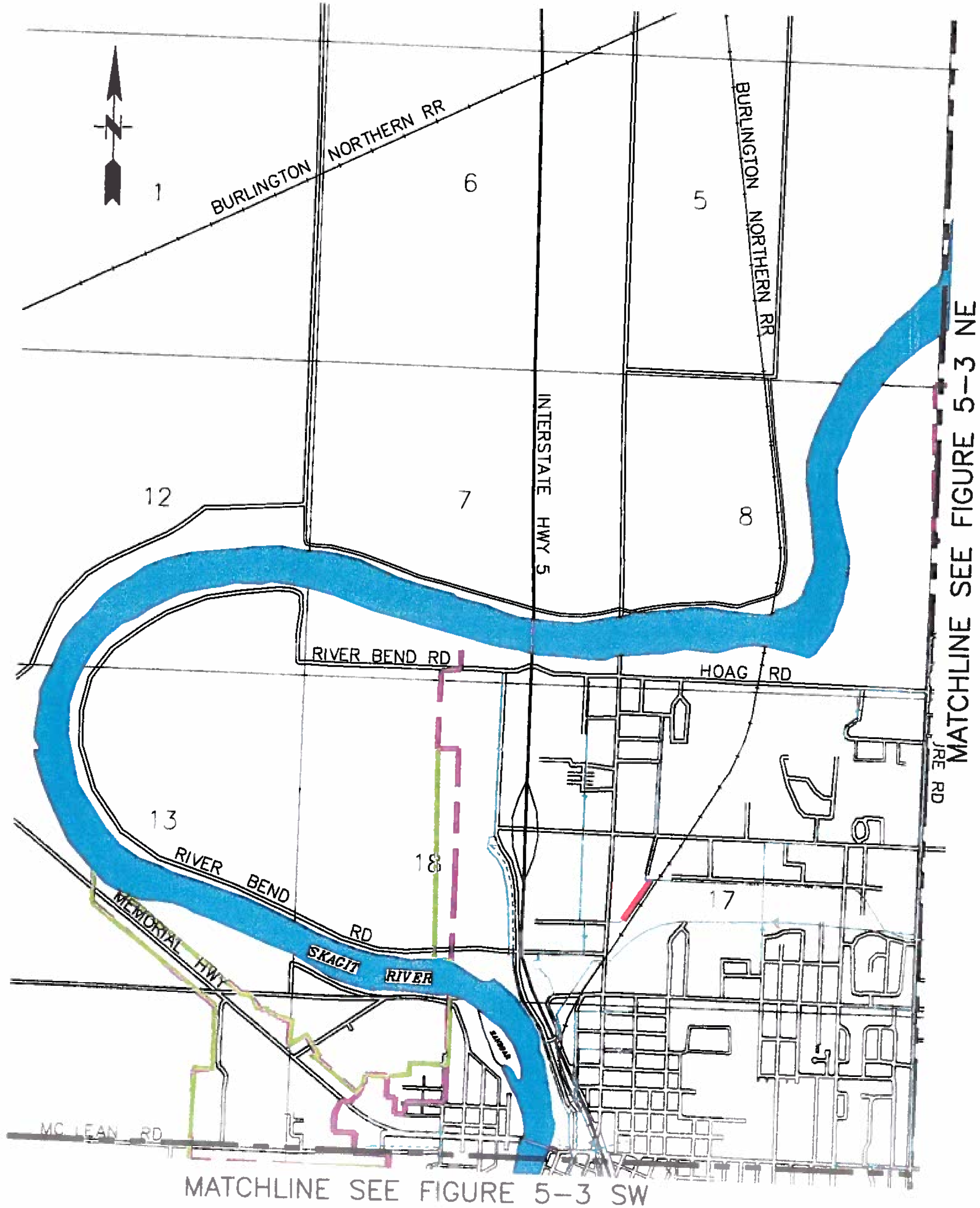
The interceptor system has lines that are predicted to approach capacity as the UGA approaches saturated development. These lines are recommended for monitoring and replacement as warranted. The following sections provide details of each of the interceptors. Table 5-2 summarizes the recommendations for 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

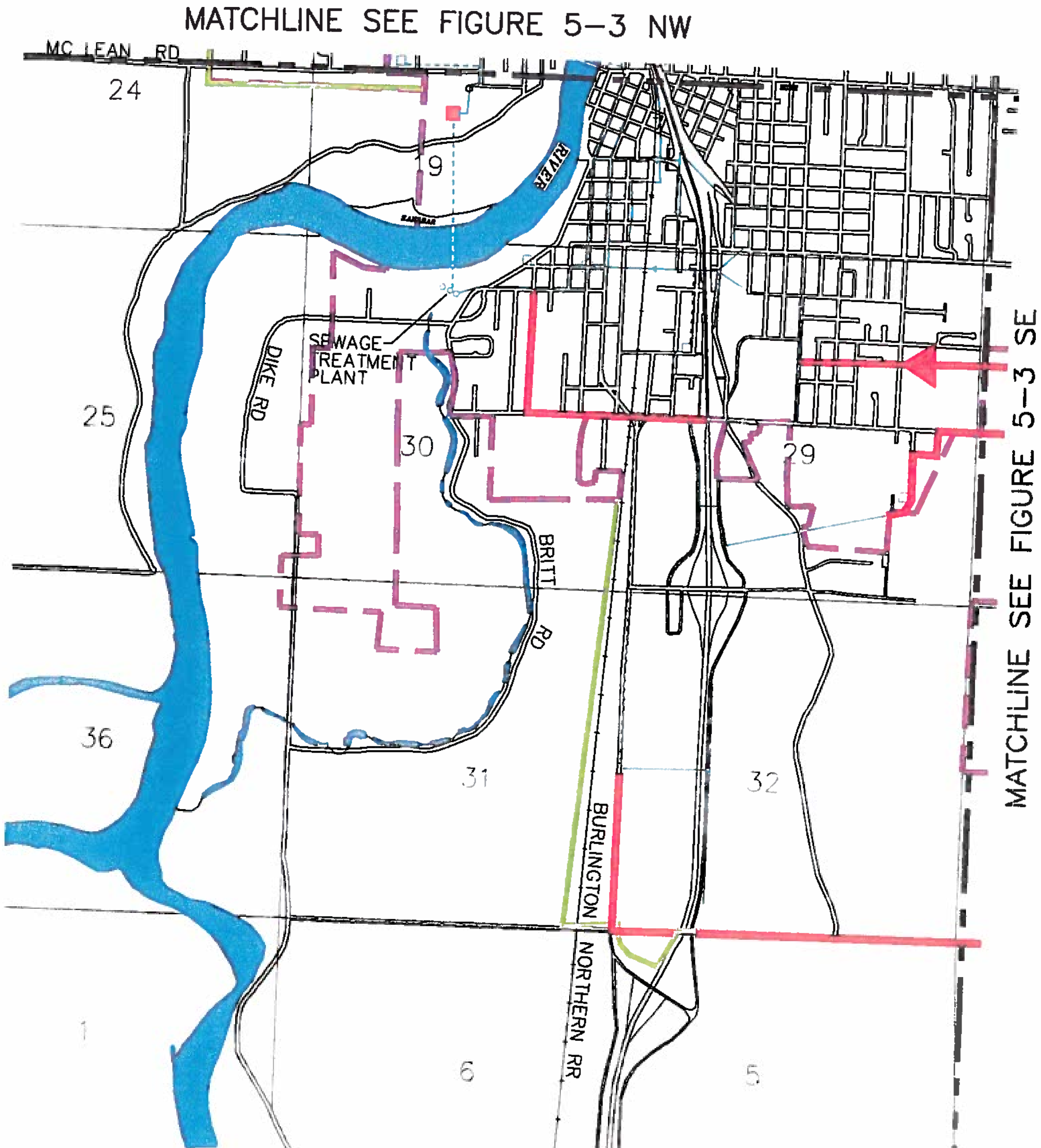
HDR Engineering, Inc.

Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: FUTURE COLLECTION SYSTEM IMPROVEMENTS

Figure No. 5-3 NW



NOTE:

1. IMPROVEMENTS SHOWN HERE ARE THE MINIMUM REQUIRED.

LEGEND:

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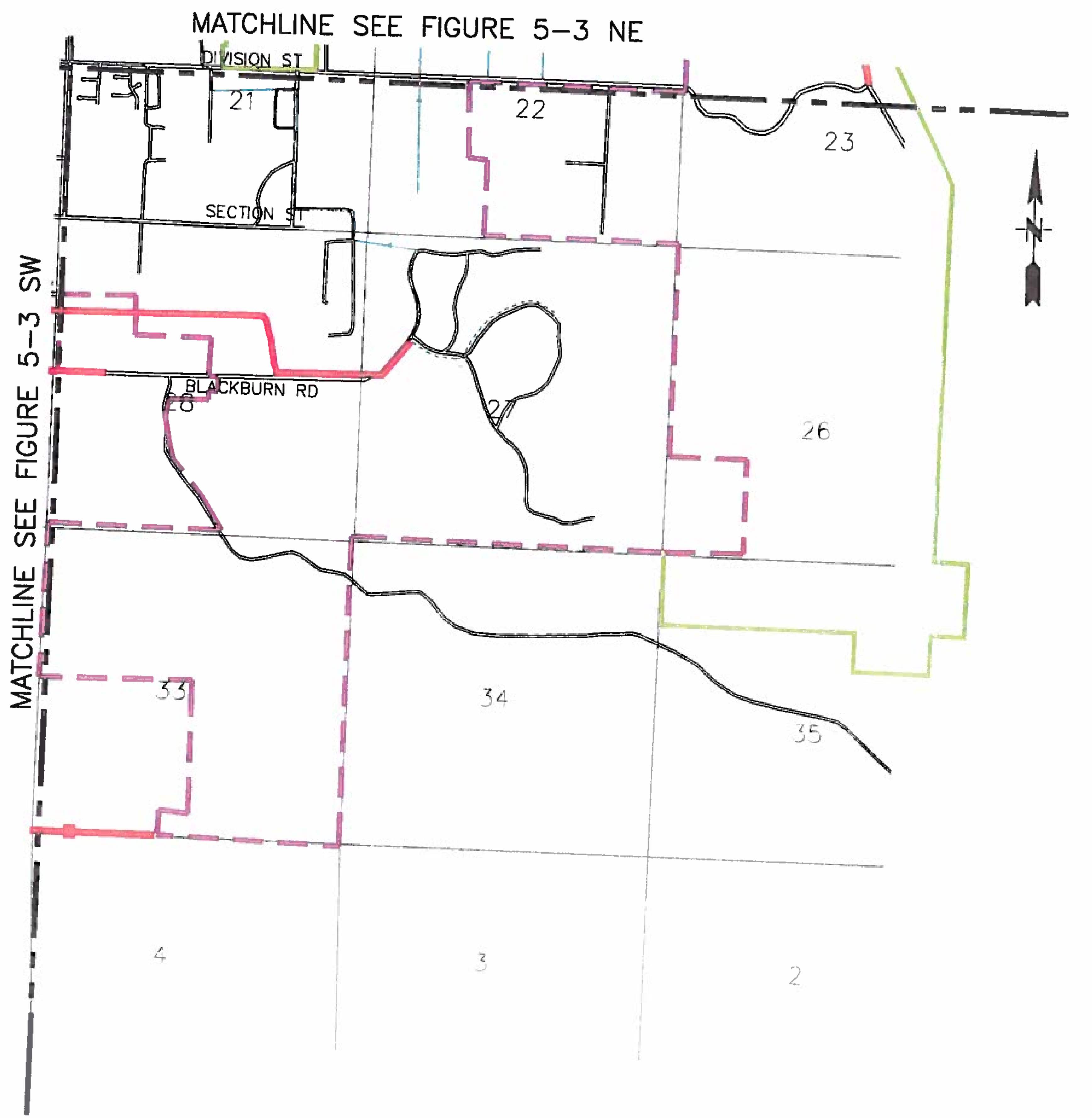
Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE	Date: FEBRUARY 2003
Sheet Title: FUTURE COLLECTION SYSTEM IMPROVEMENTS	Figure No: 5-3 SW



NOTE:
1. IMPROVEMENTS SHOWN HERE ARE THE MINIMUM REQUIRED.

LEGEND:
- FUTURE SYSTEM IMPROVEMENTS
- FUTURE FORCE MAIN
- EXISTING MAIN COLLECTION SEWERS (W/ FLOW ARROWS)
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- CITY LIMITS
- UGA BOUNDRY

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MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE	FEBRUARY 2003
FUTURE COLLECTION SYSTEM IMPROVEMENTS	5–3 SE

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.

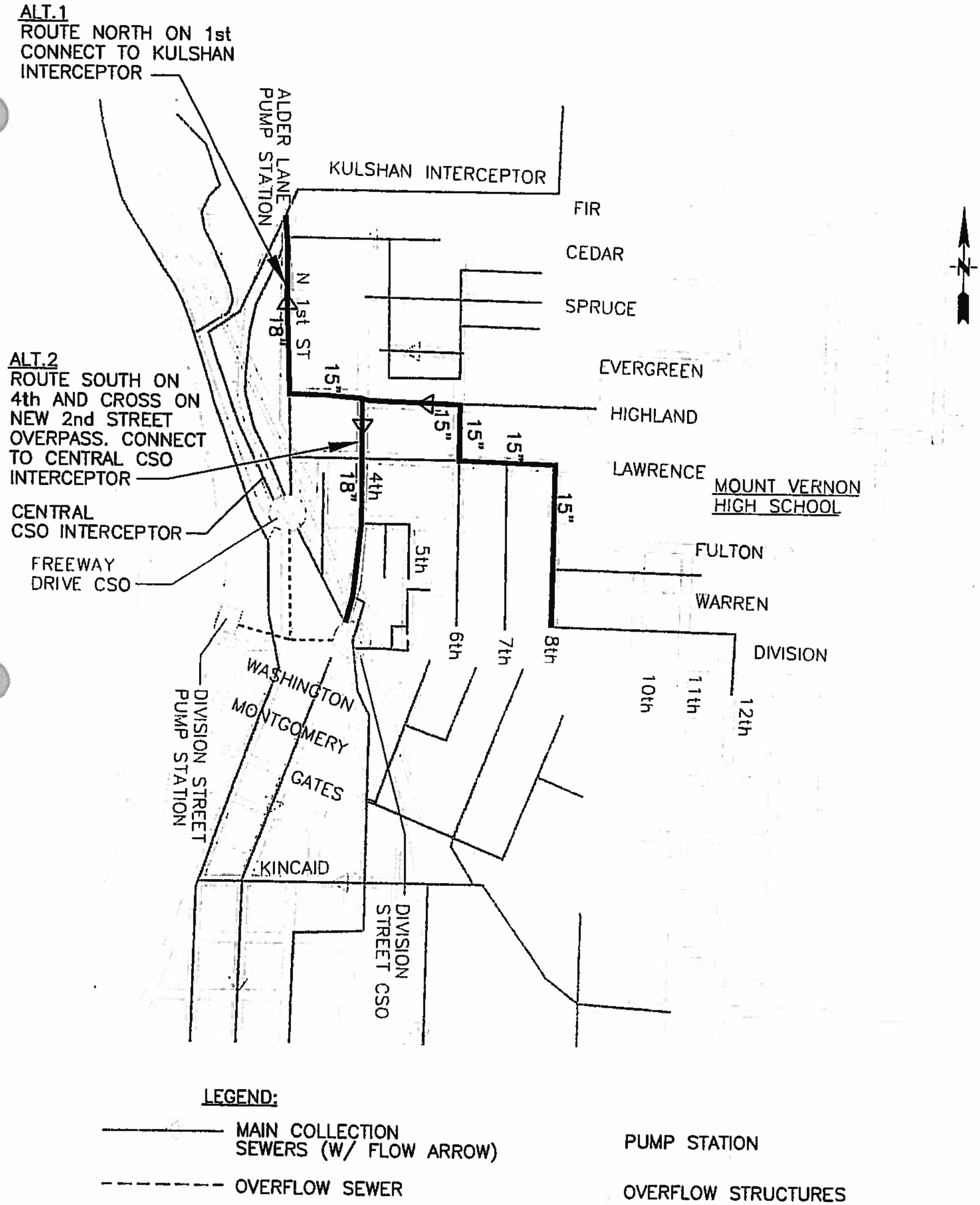
Interstate 5 Crossings		
No.	Location	Condition and Recommended Improvement
		necessary.
8.	Section Street at Wells Nursery	Condition unknown. This 16-inch provides service to only one connection, Wells Nursery. There is also a documented steady flow
9.	Park Street at South Side of Wells Nursery	Condition 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.



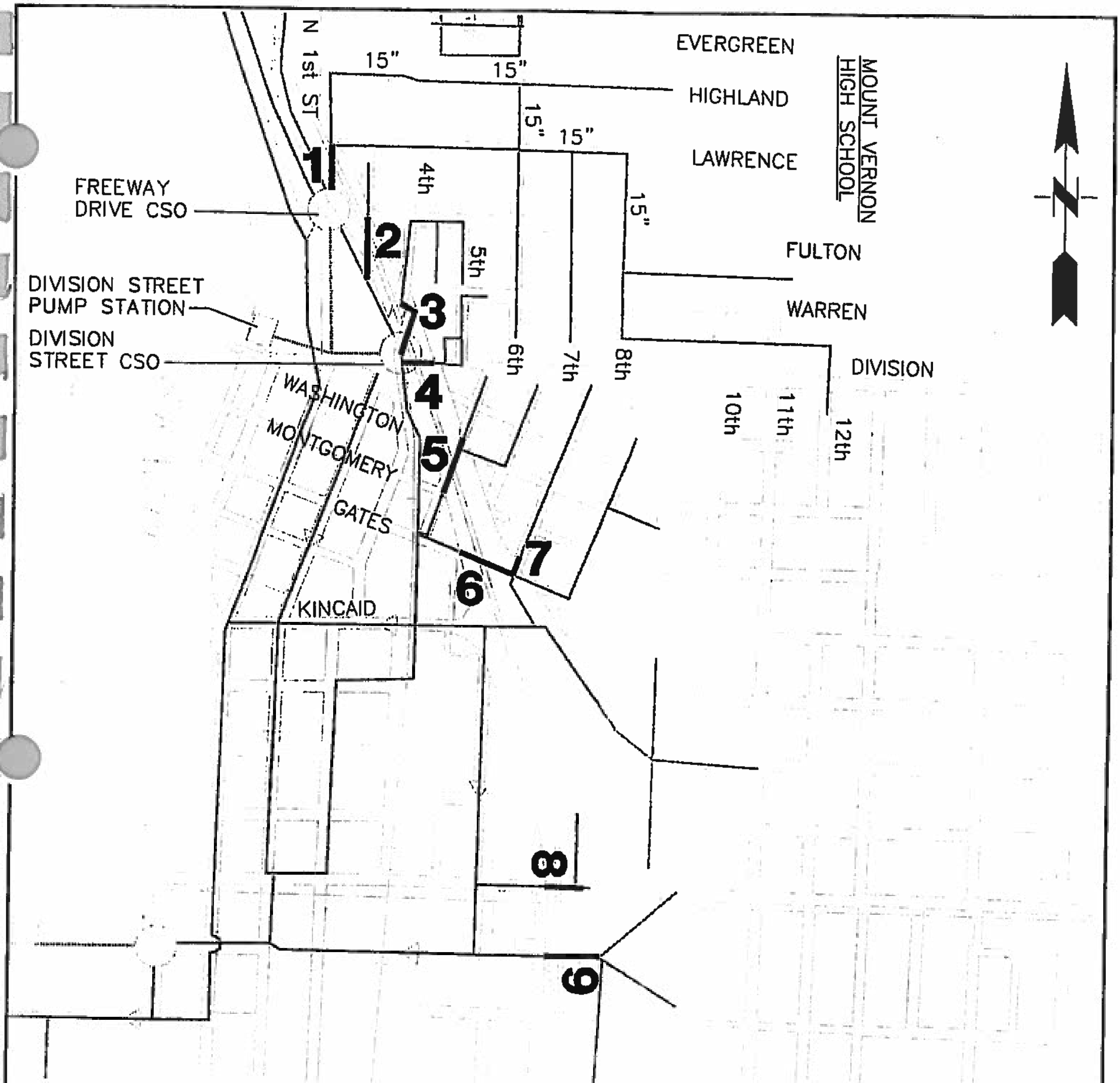
HDR Engineering, Inc.

Project Title
MOUNT VERNON COMPREHENSIVE SEWER
PLAN UPDATE

Date
FEBRUARY 2003

Sheet Title
NORTH 8th STREET IMPROVEMENTS

Figure No.
5–4



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	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	--
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 utilities is to avoid the use of gaseous chlorine when planning new facilities. Presently the City utilizes 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 1401 lbs/day	45 mg/L 2102 lbs/day
Total Suspended Solids (TSS)	30 mg/L 1401 lbs/day	45 mg/L 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.

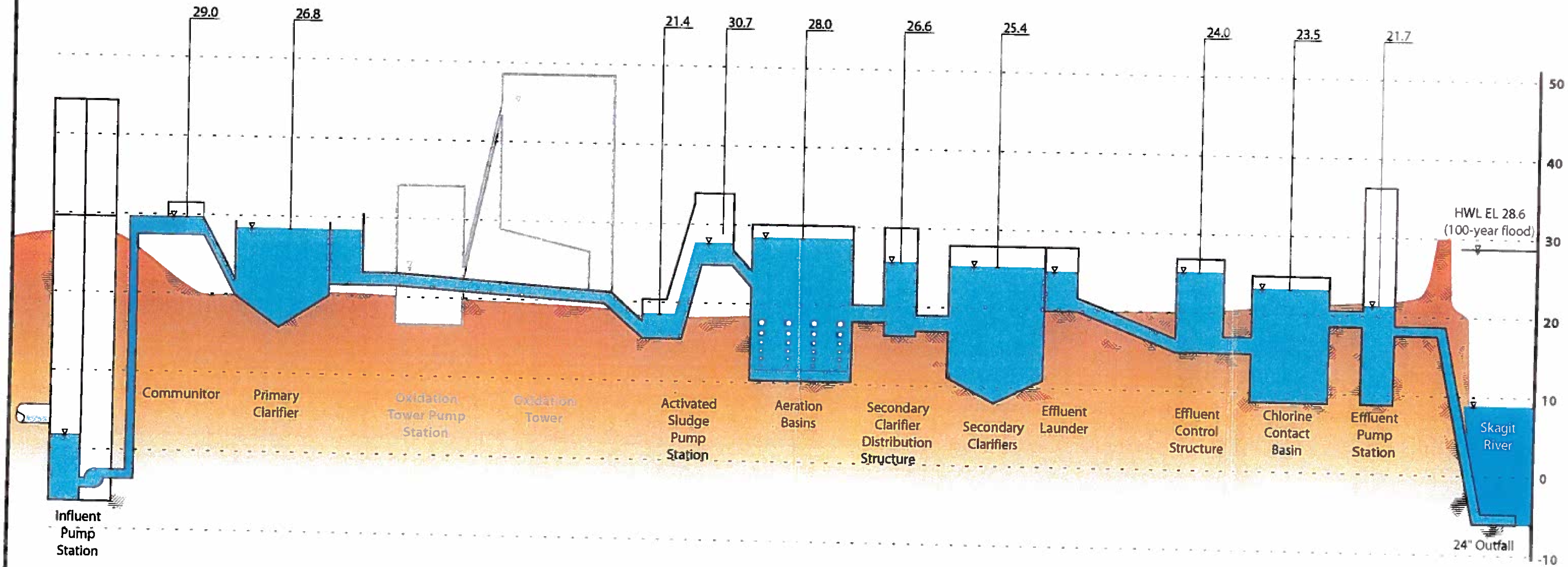
Table 7-3

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

1. Interim limit is in affect for the duration of the draft NPDES, after which time new limits of: Average Day: 9.4 μg/L, 0.44 lbs/day and Maximum Day 16.6 μg/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
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, 17.5 ft SWD	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
		0.33 MG each, 1.0 MG total	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-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 clarifiers	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 5,700 sf each	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
		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 10-ft SWD	MM	OFR	700 gpd/sf	DOE Standard	261 gpd/sf	353 gpd/sf	28.2 MGD	19.6 MGD	None	None
DAF Thickener	1- DAF Thickener	40-ft diameter 11 ft SWD 1,260 sf	MM	SLR	2.5 lb/hr-sf	DOE Standard	4.0	3.5	--	5.6 MGD	**750 sf**	**500 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, 103,400 cf (0.8 MG)	MM	SLR	140 lbVSS/kcf-d	WEF MOP8	80 lbVSS/kcf-d	70 lbVSS/kcf-d	18 MGD	13 MGD	None	None

[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.

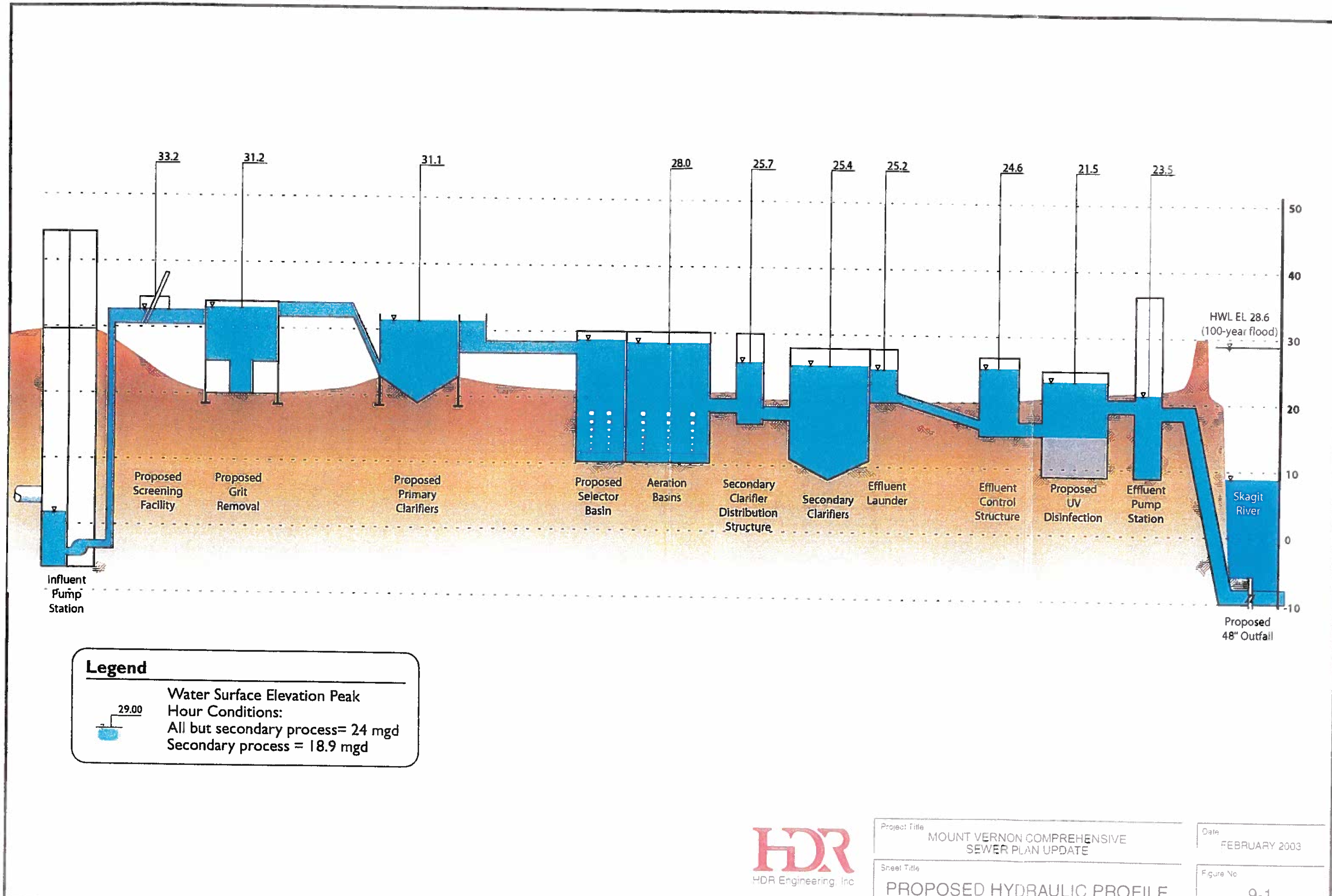
Alternative A - Existing WWTP Hydraulics

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.



33.2
31.2
31.1
28.0
25.7
25.4
25.2
24.6
21.5
23.5
50
40
30
20
10
0
-10
HWL EL 28.6
(100-year flood)
Influent Pump Station
Proposed Screening Facility
Proposed Grit Removal
Proposed Primary Clarifiers
Proposed Selector Basin
Aeration Basins
Secondary Clarifier Distribution Structure
Secondary Clarifiers
Effluent Launder
Effluent Control Structure
Proposed UV Disinfection
Effluent Pump Station
Skagit River
Proposed 48" Outfall
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)

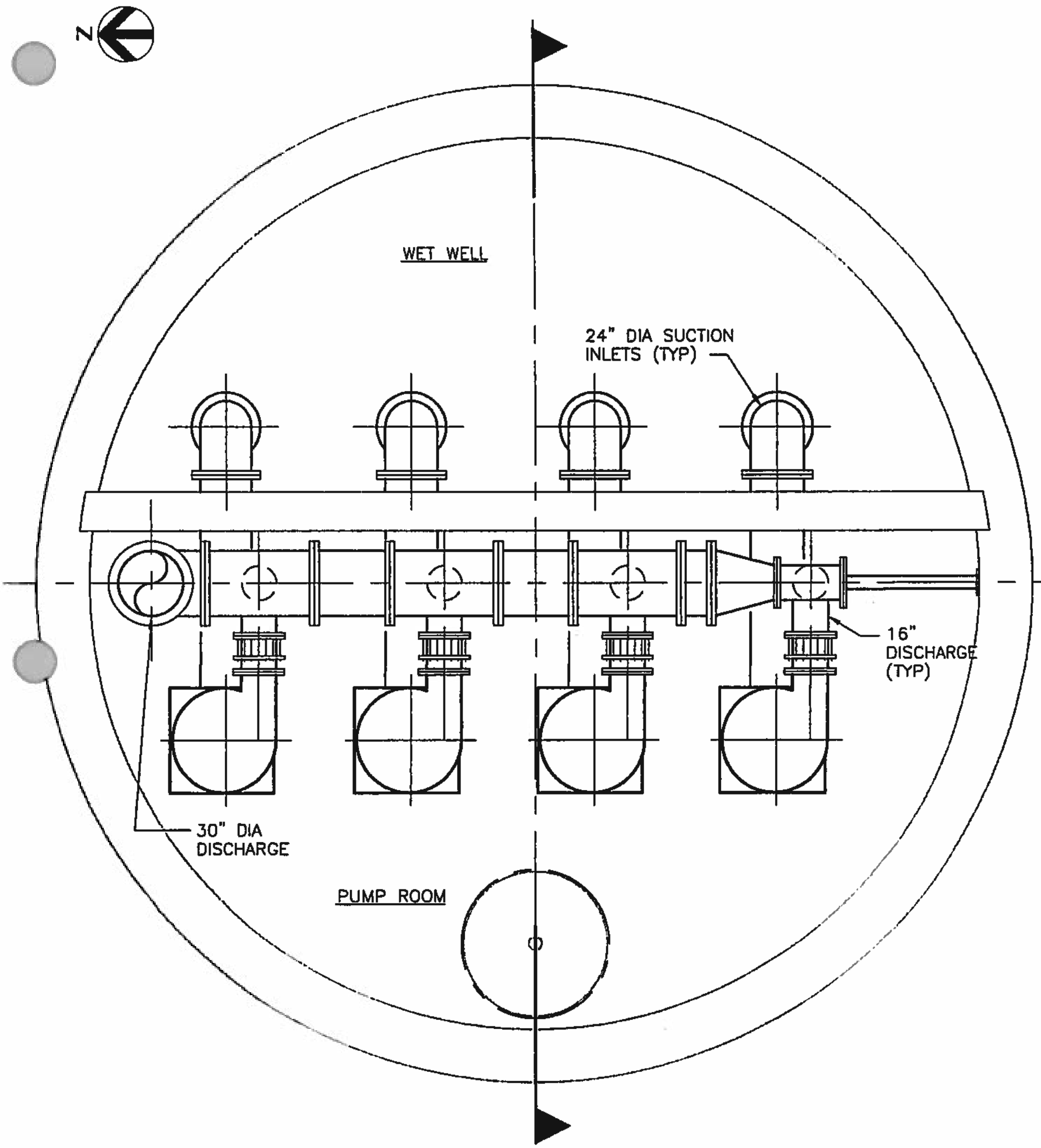
Item	Quantity	Unit	Unit Cost	Cost
Bypass Pumping	1	LS	$50,000	$50,000
Replace Existing Pumps	4	EA	$65,000	$260,000
Replace Existing Piping	1	LS	$80,000	$80,000
Forcemain	1	LS	$200,000	$200,000
Modify Existing Wetwell	1	LS	$50,000	$50,000
Replace Existing VFDs	4	EA	$40,000	$160,000
Additional Barscreen	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



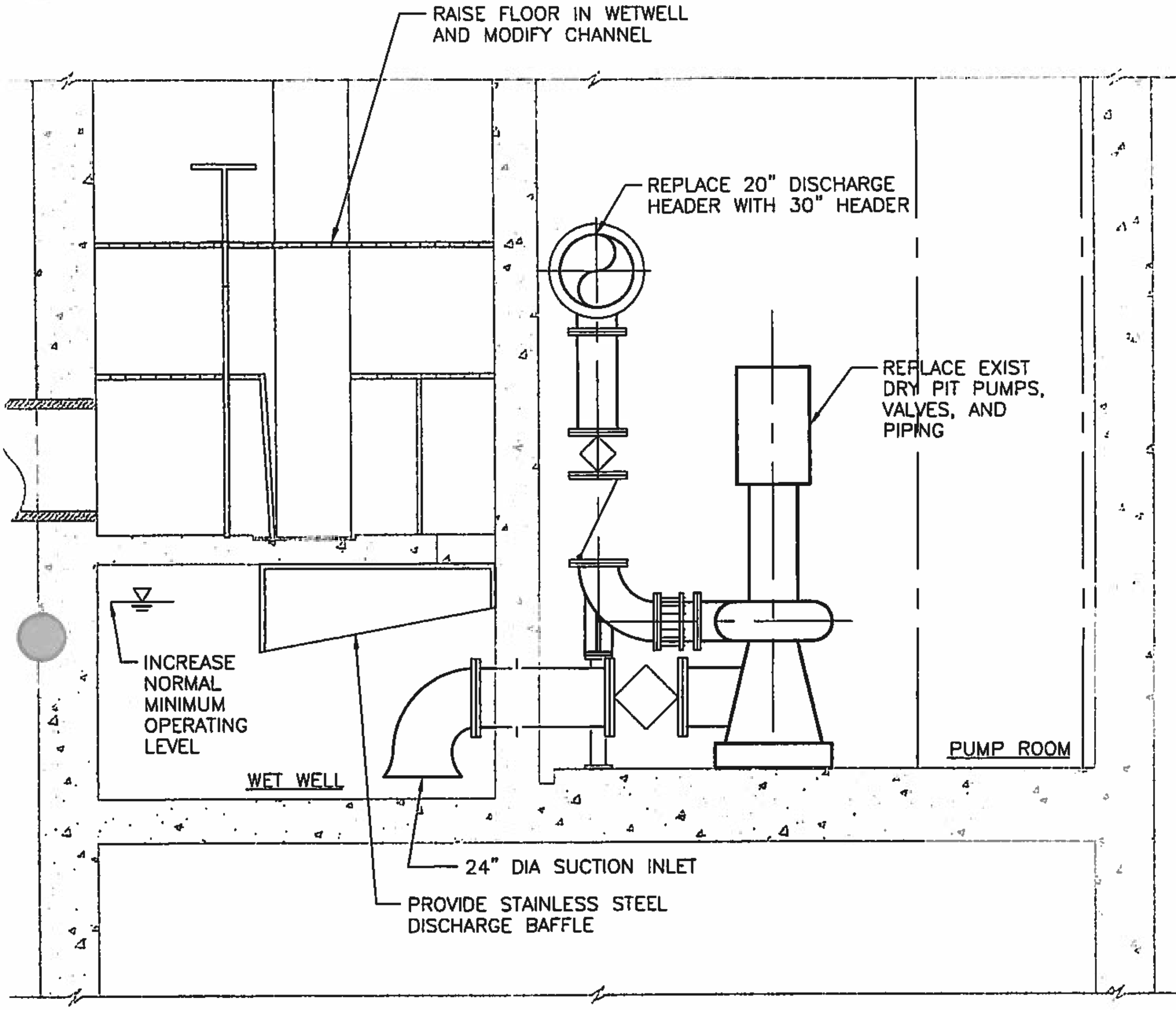
HDR Engineering, Inc.

Project Title
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date
FEBRUARY 2003

Sheet Title
INFLUENT PUMP STATION UPGRADE ALTERNATIVE A-PLAN

Figure No.
9-2



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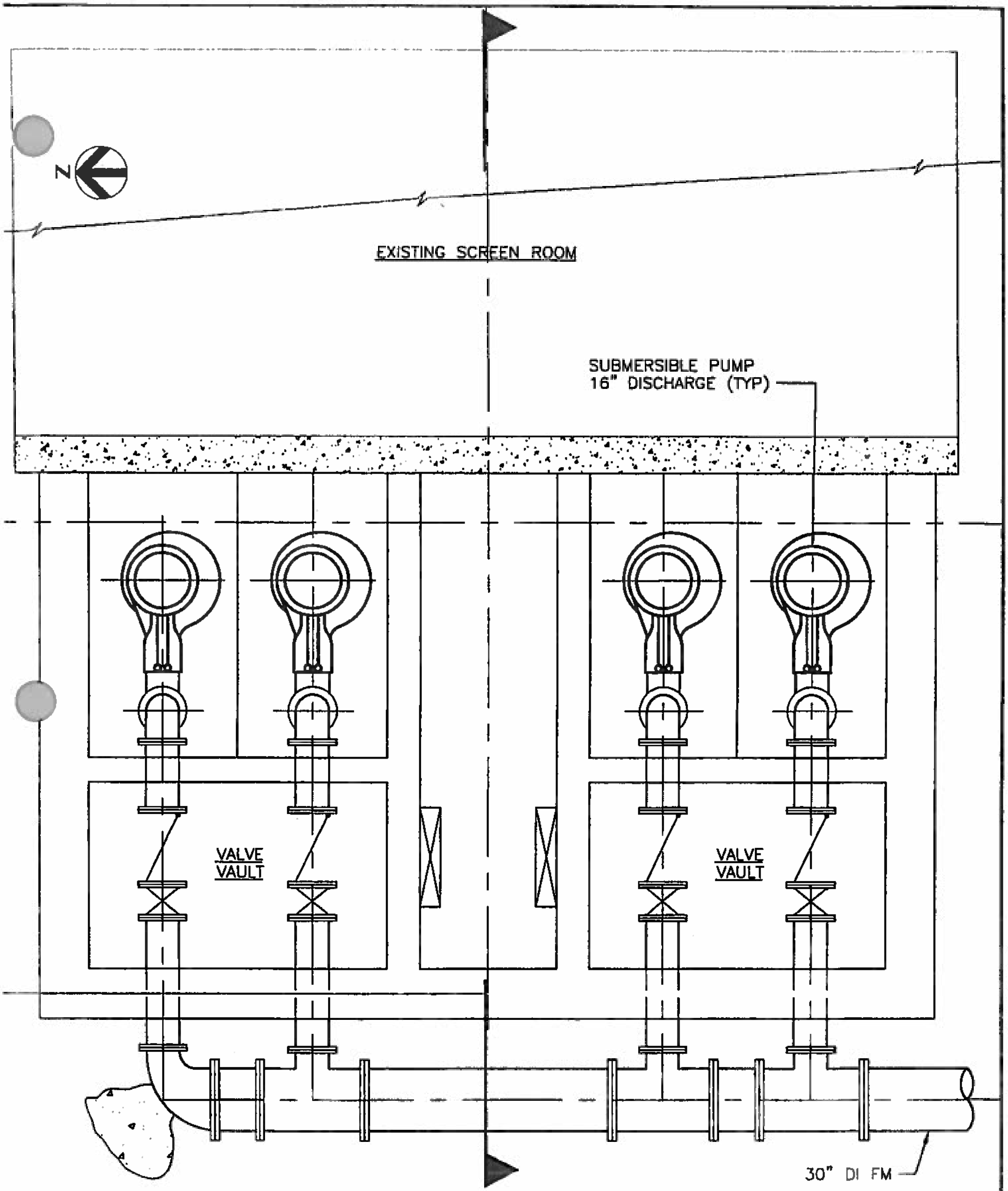
Project Title
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date
FEBRUARY 2003

Sheet Title
INFLUENT PUMP STATION UPGRADE ALTERNATIVE A–TYPICAL SECTION

Figure No.
9–3



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MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date
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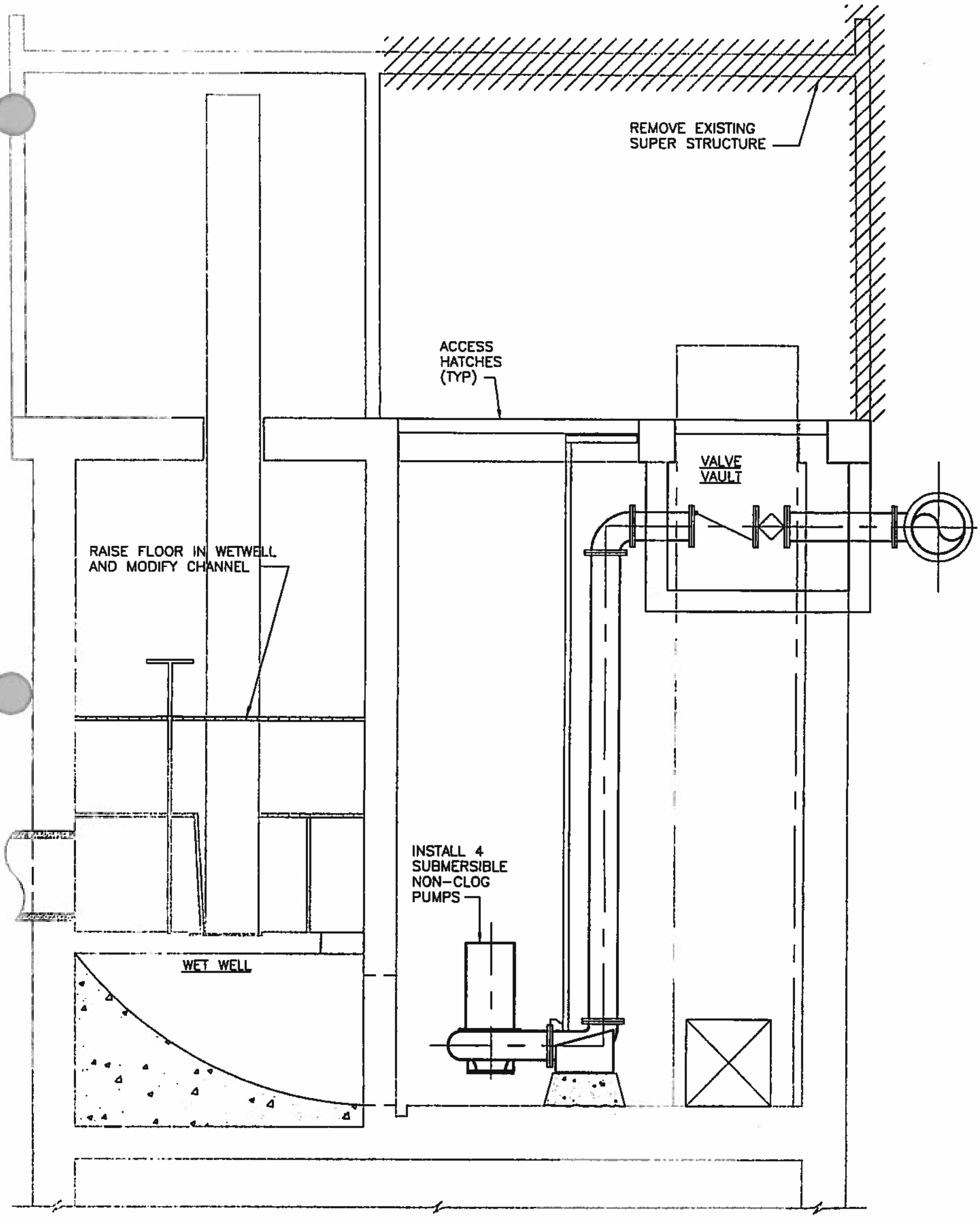
Sheet Title
INFLUENT PUMP STATION UPGRADE ALTERNATIVE B-PLAN

Figure No.
9-4



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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.

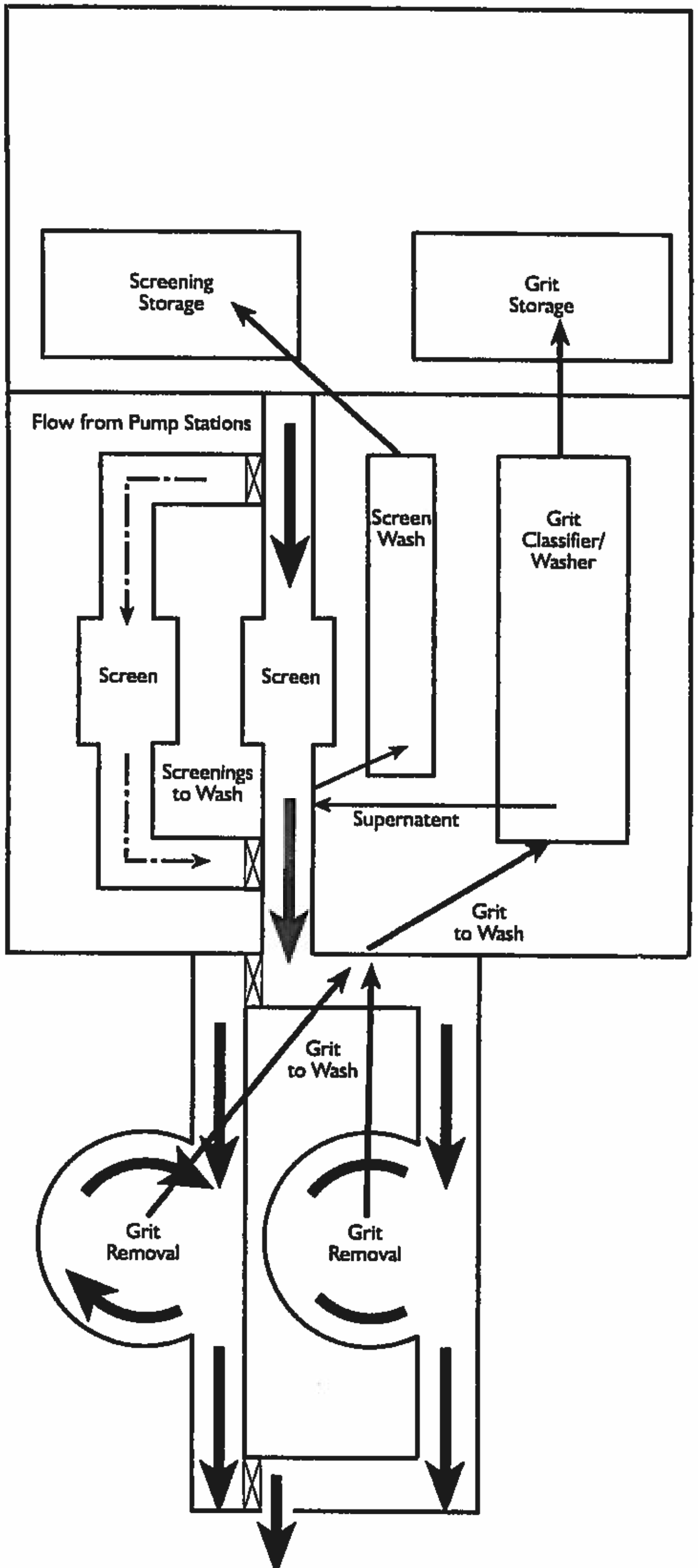
Primary Sludge and Scum Pumping

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 MOUNT VERNON COMPREHENSIVE SEWER	Date FEBRUARY 2003
Sheet Title PROPOSED HEADWORKS FACILITY	Figure No. 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.

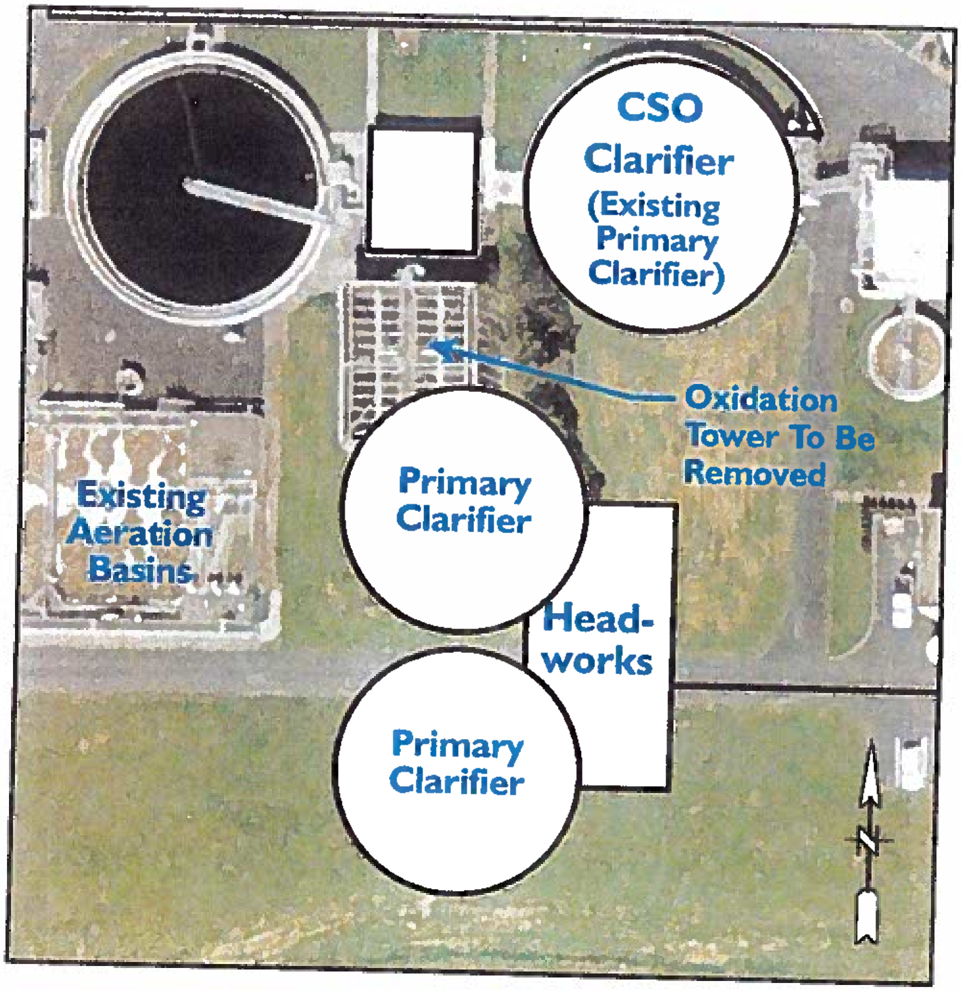
Cost Estimate

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	Date
MOUNT VERNON COMPREHENSIVE SEWER	FEBRUARY 2003
Sheet Title	**Figure No**
PROPOSED PRIMARY CLARIFIERS	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.

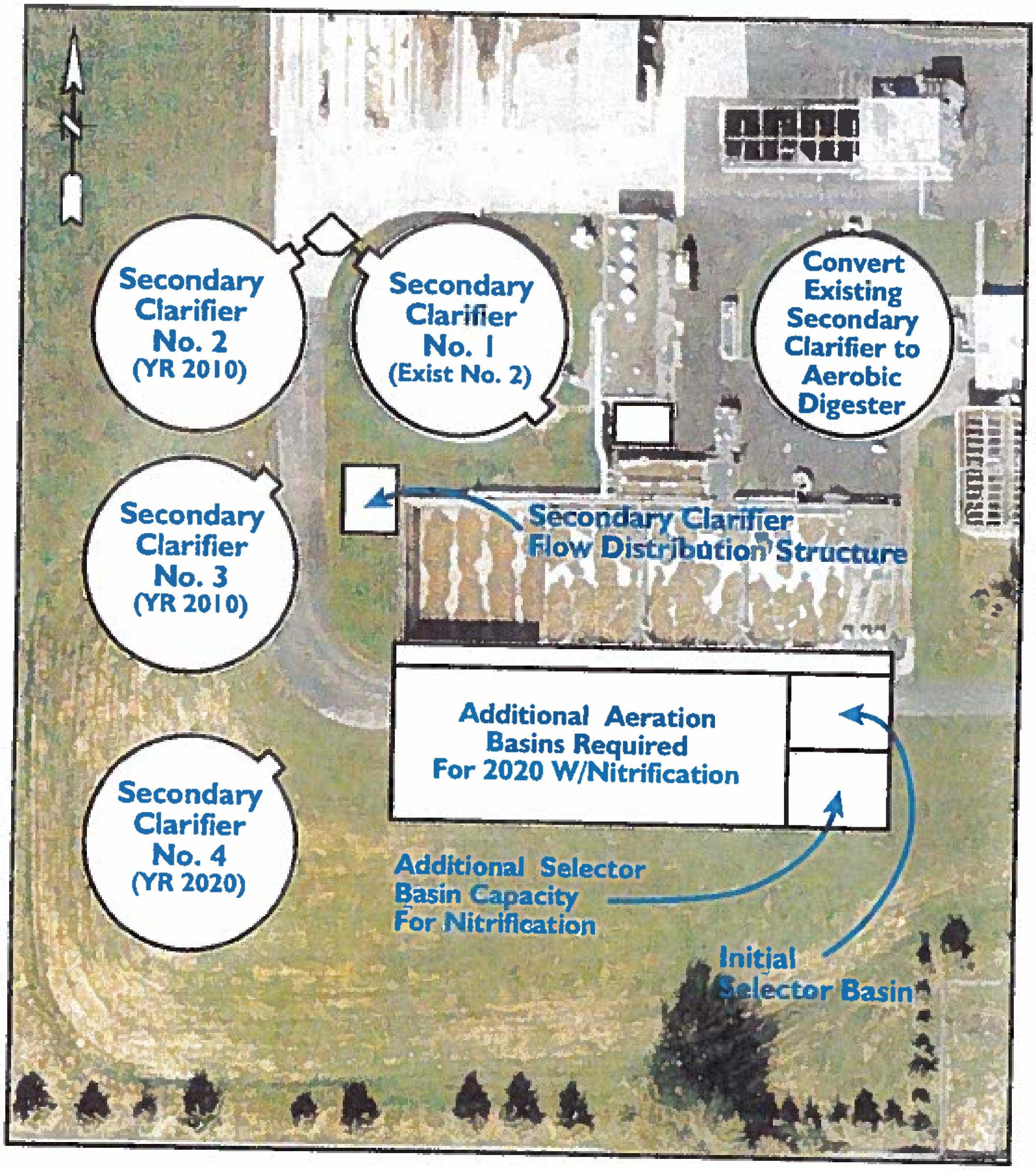
Aeration for the basins would be fine bubble, as explained in alternative A.

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.

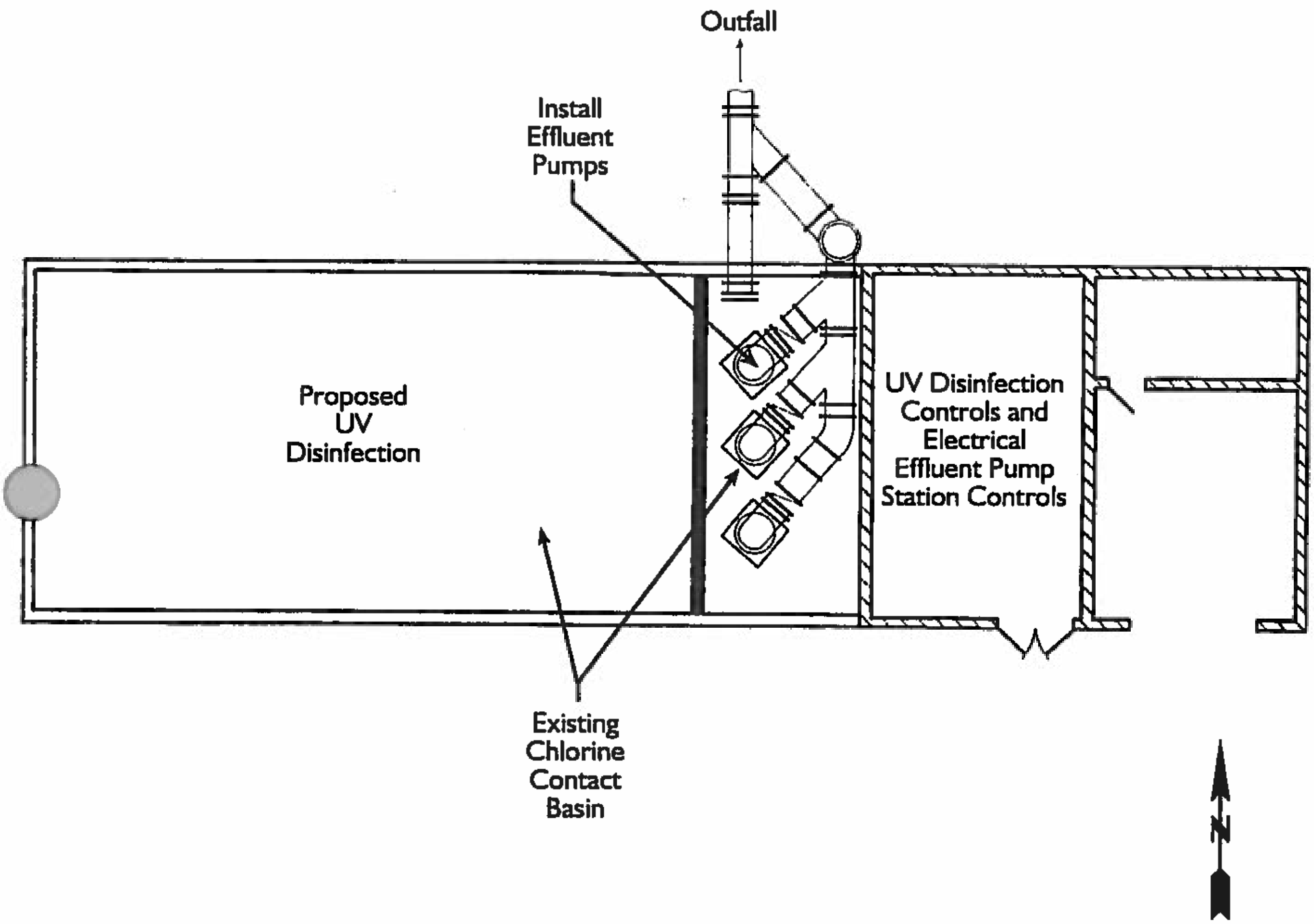
Alternative A - Retrofit Existing Effluent Pump Station

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.



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Sheet Title	**Figure No.**
UV DISINFECTION AND EFFLUENT PUMP STATION	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 • Single dilution zone	• Combined effluents would need to be addressed in future NPDES permits
Two Pipe	• Maintains option for separate CSO outfall • Lower maintenance (diffusers not required)	• Greater Capital Cost • 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 capacities 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			
Item	Co-generation Capacity		
	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.

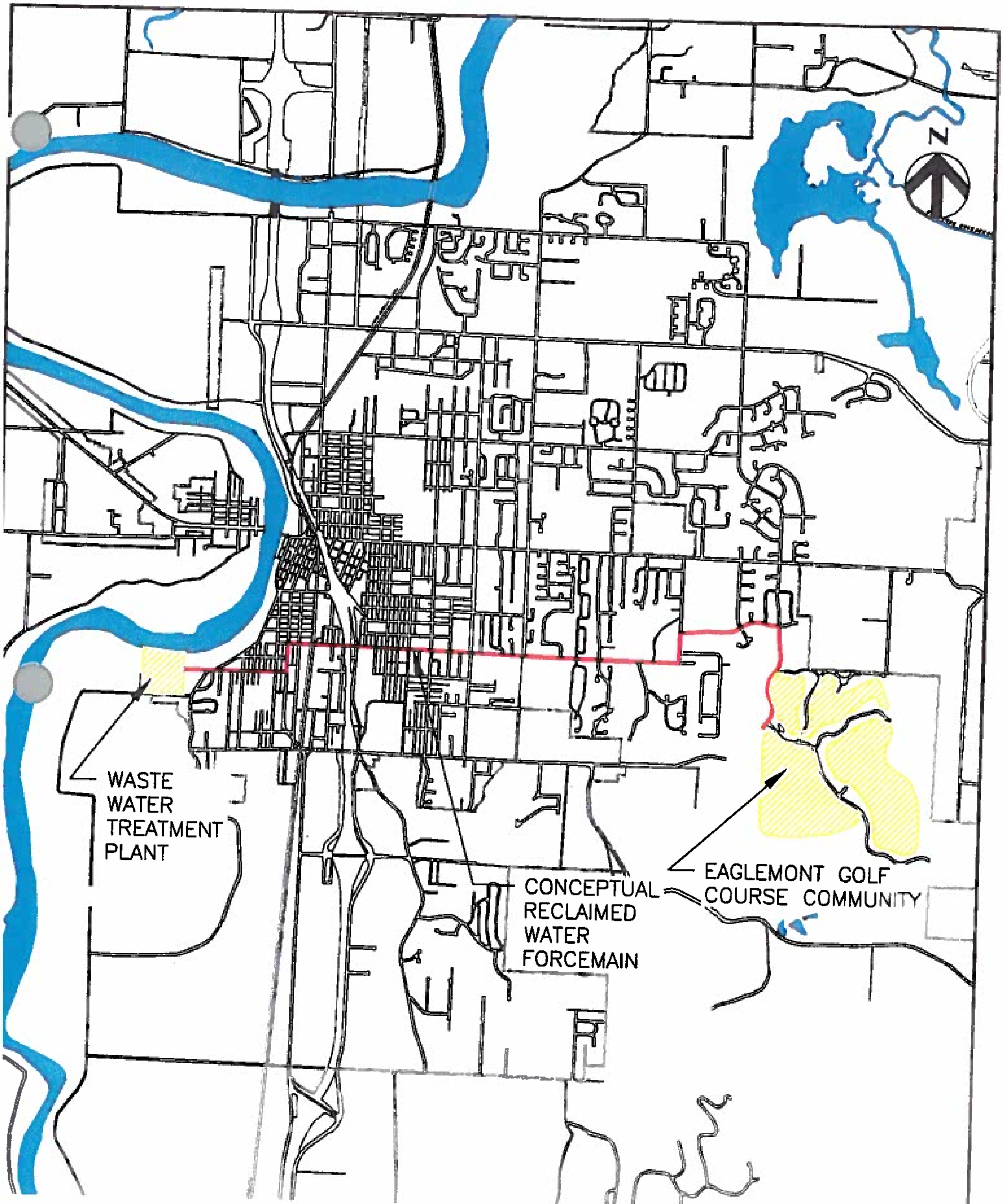
Table 9-13

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.



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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.

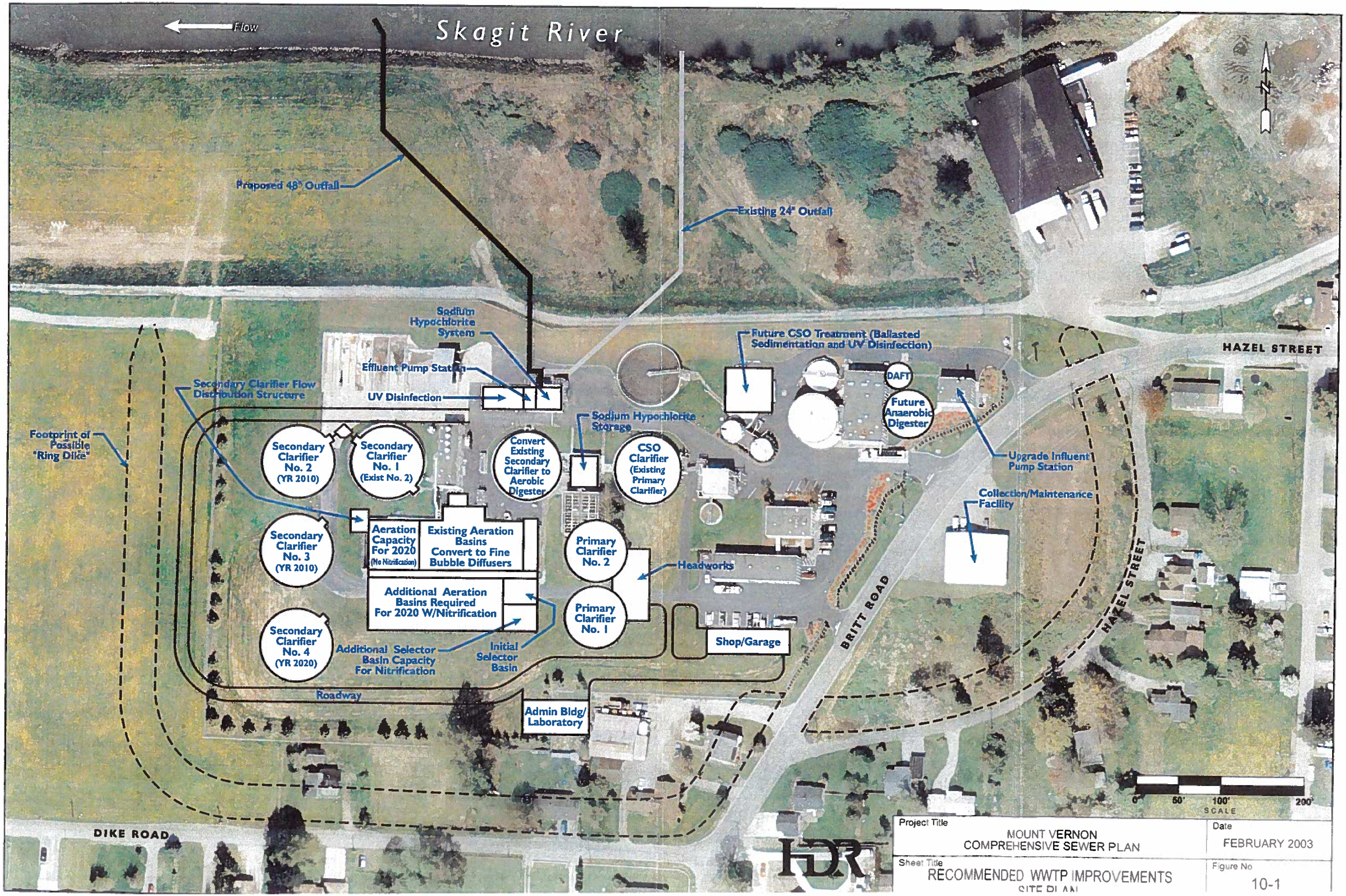
Summary of Recommendations

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

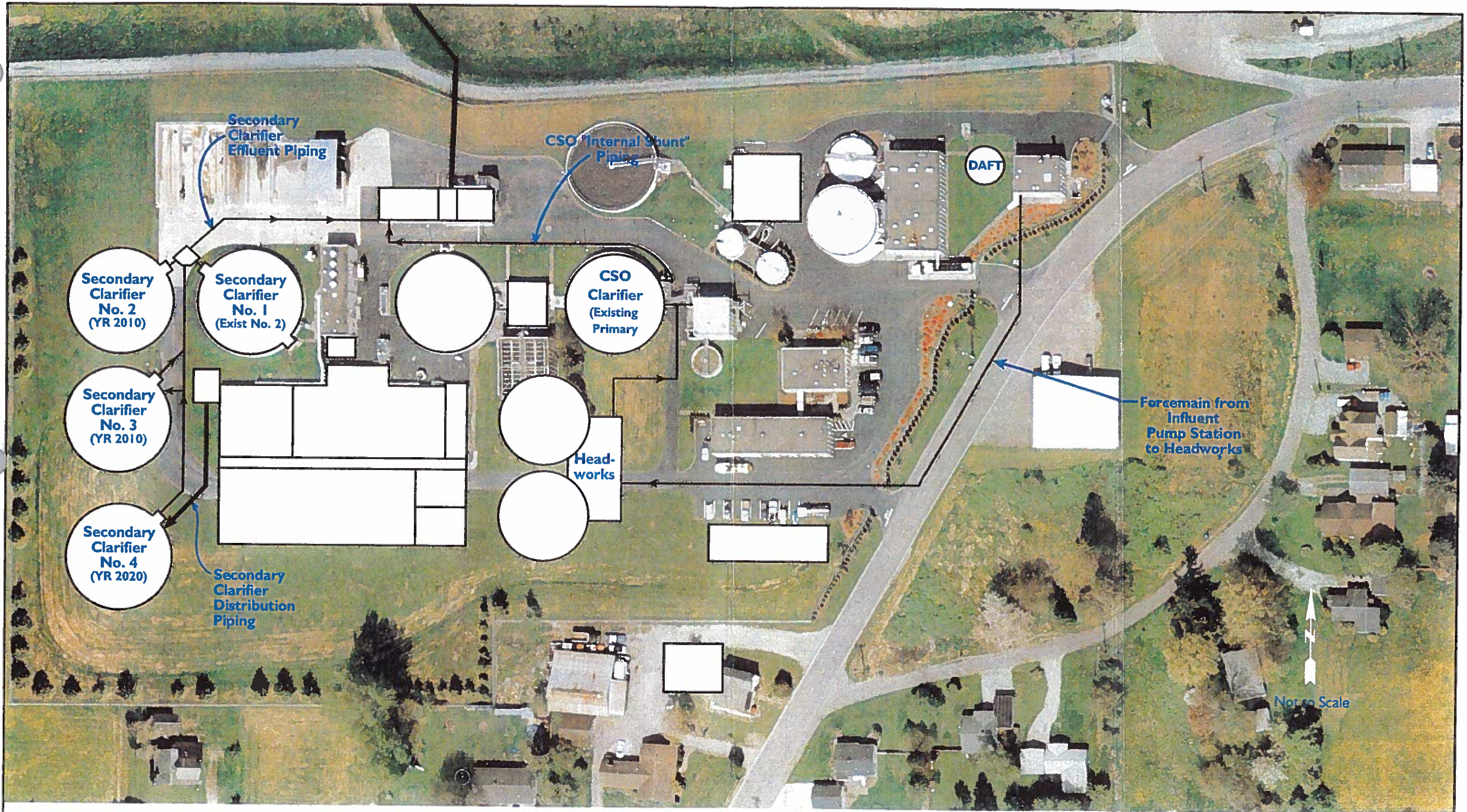
Recommended Improvements for the Wastewater Treatment Plant	
Improvement	**Capital Cost Estimate ($1,000)**
Influent Pump Station	$1,600
Headworks	$2,800
Primary Clarifiers	$1,800
Selector Basins	$600
Aeration Basins	$2,700
Chemical Feed System (pH control)	$50
Secondary Clarifiers	$3,600
UV Disinfection[2]	$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.



HDR

Project Title	Date
MOUNT VERNON COMPREHENSIVE SEWER PLAN	FEBRUARY 2003
Sheet Title	Figure No
RECOMMENDED WWTP IMPROVEMENTS SITE PLAN	10-1



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Project Title	MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE	Date	FEBRUARY 2003
Sheet Title	RECOMMENDED YARD PIPING	Figure No	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	201 1	201 2	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	201 1	201 2	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/Operation s 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	201 1	201 2	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	201 1	201 2	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	201 1	201 2	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			1,00 0												
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	201 1	201 2	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	1,00 0	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.

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City of Mount Vernon
Comprehensive Sewer Plan Amendment

April 2004

APPROVED
DEPARTMENT OF ECOLOGY
ENGINEERING MANAGEMENT
SIGNATURE
DATE 8/18/05

Prepared by

HDR Engineering, Inc.
500-108th Avenue NE, #1200
Bellevue, WA 98004

HDR

CITY OF MOUNT VERNON, WASHINGTON

COMPREHENSIVE SEWER PLAN AMENDMENT

Prepared by:	**Kenneth Hui**
Reviewed by:	**Dan Olson, John Koch**
Date:	**April 22, 2004**
Subject:	**Comprehensive Sewer Plan Amendment**
Project Number:	**09637-006070**

1.0 BACKGROUND

After the latest update to the Mount Vernon Comprehensive Sewer Plan (CSP) was approved by the Washington Department of Ecology (Ecology) in February 2003, the City of Mount Vernon requested that HDR further refine the upgrade concept proposed in the CSP.

The following guidelines were used when refining the upgrade concept:

- Minimize impact on the operations of the existing plant during upgrade construction
- Investigate the feasibility of incorporating new and emerging technologies that could improve treatment efficiencies and reliability
- Explore potential cost savings

Three configurations and several new technologies were investigated as described in the *Alternative Facility Concepts and Layouts* technical memorandum issued in July 2003. The recommendations of this study formed the basis for further evaluation. The feasibility was explored for treating peak storm flows with an innovative high rate clarification process in conjunction with the preferred modified treatment configuration. High rate clarification (HRC) has gained acceptance recently as an effective means of treating peak storm flows and minimizing combined sewer overflows (CSOs). HRC has been accepted as more municipalities pilot test the technology with promising results and as more full-scale facilities are installed throughout the country. This proposed modified treatment scheme fits into and further improves the overall long-term vision of the City of Mount Vernon as originally proposed in the 1991 Comprehensive Sewer and Combined Sewer Overflow Reduction Plan (the 1991 Plan). A separate technical memorandum summarizing the performance of the HRC process during pilot studies in the Pacific Northwest and other parts of the country was issued in November 2003. This technical memorandum is attached as Appendix A.

Subsequent correspondence and meetings with Ecology resulted in conceptual agreement by Ecology for the use of HRC treatment for flows above the maximum month level. Letters to Ecology from the City of Mount Vernon on this issue and related meeting minutes are attached as Appendix B.

A public hearing was held on January 30, 2004 to inform the public about the modified treatment scheme, which would result in an amendment to the City Comprehensive Plan. A copy of the public hearing notice and the project narrative are included as Appendix C.

The purpose of this technical memorandum is to document proposed changes to the upgrade concept for the primary and secondary liquid-stream treatment processes at the City of Mount Vernon Wastewater Treatment Plant. This memorandum focuses on HRC as a new technology and its associated benefits to the City of Mount Vernon.

1.1 Sections Affected in the Comprehensive Sewer Plan Update

The following sections in the February 2003 Comprehensive Sewer Plan are affected by this amendment:

- The 'Combined Sewer Overflow' section, 'Wastewater Treatment Facility Improvements' section, Figure ES-1, Table ES-1, and Table ES-4 of the Executive Summary
- The 'Projected Flows and Loads' section, and Tables 3-5 through 3-9 of Chapter 3 – Basic Planning Data
- The 'Recommended CSO Reduction Alternative' section, Table 4-4 and Table 4-5 of Chapter 4 – Combined Sewer System
- The 'Capacity Analysis' section, Tables 8-3 through 8-5 of Chapter 8 – Wastewater Treatment Plant Analysis
- The 'Primary Clarifiers' section, the 'Activated Sludge Process' section, Figure 10-1, Figure 10-2, and Table 10-1 of Chapter 10 – Recommended WWTP Alternatives
- Table 11-1 and Table 11-2 of Chapter 11 – Capital Improvement Plan

2.0 REVISED PROJECTED FLOWS AND LOADS

Projected flows and loads developed in the CSP were further refined during the Predesign Phase based on recent growth rate. The refined projected flows and loads values served to provide the basis of a BIOWIN computer simulation, to allow further evaluation of the treatment process capacity, and to provide a means of evaluating upgrade options. This section replaces the 'Projected Flows and Loads' section and Tables 3-5 through 3-9 of Chapter 3 – Basic Planning Data of the CSP.

2.1 Revised Projected Flow

Table 1 shows the revised projected flow.

Table 1: Revised Projected Flow (New CSP Table 3-5)

	2010	2015	2020	Build-Out
Average Wet Weather Flow, mgd	4.35	5.14	6.07	10.50
Maximum Month Average Design Flow, mgd	7.26	8.57	10.14	17.80
Peak Daily Flow, mgd	11.40	13.47	15.90	26.78
Peak Hour Flow, mgd	17.00	25.90	25.90	45.10

2.2 Revised Projected Loads

Tables 2 through 4 show the revised projected biochemical oxygen demand (BOD), total suspended solids (TSS), and ammonia loads, respectively.

Table 2: Revised Projected Biochemical Oxygen Demand (BOD) Loads (New CSP Table 3-6)

	2010	2015	2020	Build-Out
Average Annual, pounds per day (lb/d)	6,566	7,759	9,163	15,850
Maximum Month, lb/d	8,356	9,863	11,670	20,486
Peak Daily Flow, lb/d	14,261	16,847	19,895	33,502
Peak Hour Flow, lb/d	14,261	16,847	19,895	33,502

Table 3: Revised Projected Total Suspended Solids (TSS) Loads (New CSP Table 3-7)

	2010	2015	2020	Build-Out
Average Annual, pounds per day (lb/d)	6,240	7,373	8,707	15,062
Maximum Month, lb/d	8,840	10,435	12,347	21,674
Peak Daily Flow, lb/d	13,501	15,948	18,834	31,715
Peak Hour Flow, lb/d	13,501	15,948	18,834	31,715

Table 4: Revised Projected Ammonia Loads (New CSP Table 3-8)

	2010	2015	2020	Build-Out
Average Annual, pounds per day (lb/d)	762	900	1,063	1,839
Maximum Month, lb/d	951	1,122	1,328	2,331
Peak Daily Flow, lb/d	1,046	1,235	1,459	2,456
Peak Hour Flow, lb/d	1,046	1,235	1,459	2,456

3.0 MODIFICATION TO CSO REDUCTION PLAN

This section replaces The 'Recommended CSO Reduction Alternative' section, Table 4-4 and Table 4-5 of Chapter 4 – Combined Sewer System in the CSP.

3.1 Long-Term Vision of CSO Reduction Plan

On April 11, 1996, an Order on Consent was issued by the Washington Department of Ecology and signed by the City of Mount Vernon to implement the vision of the CSO Reduction Plan. The vision of the CSO Reduction Plan is to provide a combination of storage facilities and peak storm flow treatment facilities to achieve the greatest reasonable reduction of CSOs. This means limiting the frequency of untreated CSOs to an average of no more than one per year, per requirement of Chapter 90.48.480 of the Revised Code of Washington (RCW).

The CSO Reduction Plan proposed in the 1991 Plan consisted of a 3-phase approach. In Phase 1, it was proposed that inline storage be provided for CSO flows that would have been discharged to the Skagit River. Stored CSO flows are conveyed to the wastewater treatment plant (WWTP) for treatment and disposal as capacity allows. The City concluded Phase 1 of the CSO Reduction Plan when the Central CSO Regulator was completed and put online in December 1997. The Central CSO Regulator has a volume of close to 1 million gallons and the storage capacity ranges between 0.6 and 0.8 million gallons, depending on the volume used in conveyance of storm flow. Installation of the Central CSO Regulator was projected to reduce overflows to 12 events per year; however, the overflows event from 1998 to 2003 has averaged 8 per year.

Phase 2 of the originally proposed CSO Reduction Plan consists of upgrading the WWTP treatment capacity to accommodate the combined maximum conveyance capacity (25.8 mgd) of the Hazel Street Interceptor and the West Mount Vernon Pump Station. A total of 18.3 mgd would receive full secondary treatment after building of new aeration basins, while 7.5 mgd of peak storm flow would be treated separately in a CSO clarifier. The existing primary clarifier would be converted to a CSO clarifier after new primary clarifiers are constructed upstream of the upgraded secondary treatment units. Details of the original CSO and WWTP secondary treatment upgrade are discussed in Section 3.2 and Section 4.1, respectively.

Innovative HRC technology for treating peak storm flows has recently matured, and the Central CSO Regulator has been successful in reducing CSOs to a level beyond the original expectation. In light of these factors, the CSP amendment seeks to implement this technology at Phase 2 with the goal of successfully reducing CSOs to less than one untreated event per year during this phase. By maximizing capacity of the existing aeration basin and utilizing the enhanced performance of two proposed 6 mgd HRC units, it would be possible to achieve a lower pollutant loading to the Skagit River with the revised treatment scheme. Details of the proposed changes to the CSO upgrades, proposed changes to the WWTP secondary treatment upgrades, and mass balance analyses are presented in Section 3.3, Section 4.2, and Section 5.0, respectively.

It was proposed in the 2003 Comprehensive Sewer Plan Update that HRC technology be implemented during Phase 3 to increase the combined treatment capacity of the WWTP to 48 mgd; this would reduce CSOs to less than one untreated event per year. Depending on the performance of the revised Phase 2 upgrade, the number and capacities of extra HRC units may be further revised in the future. At present, it is proposed that two more 6-mgd HRC units be installed, bringing the total to four units with a total HRC capacity of 24 mgd.

Monitoring the performance of the Central CSO Regulator system through 2010 will provide an additional six years of flow data. The data collected will be used to further refine the CSO compliance alternative and optimize the ultimate execution of the Phase 2 and Phase 3 CSO Reduction Plan.

3.2 CSO Reduction Plan Improvement Proposed in the Comprehensive Sewer Plan

In the February 2003 Comprehensive Sewer Plan Update, the following upgrades were proposed for installation between 2004 and 2009:

- The Influent Pump Station would be upgraded to handle a flow of 24 mgd. The West Mount Vernon Pump Station would be upgraded to handle a flow of 1.8 mgd. The peak flow to the wastewater treatment plant would be 25.8 mgd.
- The new headworks (fine screens and degritting) would be sized to handle the combined peak flow of 25.8 mgd.
- Two new 75-foot-diameter primary clarifiers would be built to handle a peak flow of 9.2 mgd each (18.3 mgd total). The primary effluent would then be treated by the upgraded secondary treatment process (18.3 mgd capacity).
- The existing primary clarifier would be converted to a CSO clarifier for CSO treatment via the internal shunt mechanism to treat the remaining 7.5 mgd flow that would not be treated by secondary treatment.
- Effluent from the secondary treatment process (18.3 mgd) and the CSO clarifier (7.5 mgd) would be blended at the disinfection facility for disinfection and disposal via a common outfall.

The following upgrades were proposed for the next and final phase of the CSO Reduction Plan in 2013, as described in Chapter 4 of the Comprehensive Sewer Plan Update:

- Park Street Pump Station would be upgraded to separately convey CSO and storm flow.
- CSOs would be pumped to the WWTP via a new CSO force main to a new HRC system for treatment. The proposed peak hour HRC treatment capacity was 22.2 mgd.
- The HRC effluent would be disinfected by an ultraviolet (UV) disinfection system and pumped to the treatment plant outfall for final disposal.

3.3 Proposed Modification to CSO Reduction Plan Improvement

The following modifications to the CSO Reduction Plan were proposed:

- Two 6 mgd HRC modules would be installed at the WWTP to handle flows above the maximum capacity of the secondary treatment process. It is proposed that the HRC modules be installed prior to January 1, 2015. The combined treatment capacity of the upgraded system would be sufficient to handle the projected 2020 peak hour flow of 25.9 mgd.

- The HRC effluent would be disinfected and blended with the disinfected secondary effluent before final disposal via the treatment plant outfall.
- Beyond 2020, two more HRC modules could be installed at the WWTP, together with other improvements in the liquid-stream treatment, to provide a combined treatment capacity sufficient to handle the projected build-out peak hour flow of 45.1 mgd.

4.0 MODIFICATIONS TO LIQUID-STREAM TREATMENT UPGRADE

This section replaces the following sections in the CSP: the 'Primary Clarifiers' section, the 'Activated Sludge Process' section, Figure 10-1, Figure 10-2, and Table 10-1 of Chapter 10 – Recommended WWTP Alternatives.

4.1 Liquid-Stream Treatment Upgrade Proposed in the Comprehensive Sewer Plan

The upgrades to the Influent Pump Station, West Mount Vernon Pump Station, headworks, and primary treatment proposed in the February 2003 Comprehensive Sewer Plan Update were summarized in Section 3.1. Proposed improvements to the secondary treatment process include:

- New selector basins for filament control would be added in two phases. The selector basins would be operated in aerobic mode in the first phase to accommodate the non-nitrifying mode of the plant. In the second phase, additional tanks would be installed and the selector basins would be operated in anoxic mode to accommodate the anticipated nitrifying mode of the plant.
- The existing 0.5 million gallon Aeration Basin No. 4 would be used for secondary treatment instead of for waste activated sludge (WAS) storage.
- The existing coarse bubble diffusers would be replaced with fine bubble membrane disc diffusers to improve aeration efficiency.
- An additional 1.2 million gallons of aeration basin tankage would be added in the second phase improvement. This would allow the secondary treatment to achieve full nitrification, reducing the occasional ammonia peak from solids treatment internal recycle, and thus complying with the discharge permit requirement.
- The existing Secondary Clarifier No. 1 would be taken offline and would be converted to WAS storage.
- Two new 85 foot diameter clarifiers and distribution structure would be added by 2010. A third new 85 foot diameter clarifier would be added by 2020.

4.2 Proposed Modification to Liquid-Stream Treatment Upgrade

Based on the revised projected flows and loads, several BIOWIN computer simulations were conducted to: (a) analyze existing secondary treatment system capacity, and (b) evaluate options for upgrading the existing secondary process to provide sufficient capacity for the projected future flows and loads.

The following changes are proposed:

- Sufficient fine-screen capacity would be installed to handle the projected 2020 peak hour flow of 25.9 mgd. An additional channel would be constructed for installation of a third screen to handle build-out peak flows.
- Flow splitting to HRC would be installed upstream of the degritting system. The high rate clarification process manufacturers indicate that degritting is not required upstream of the HRC.

- Degritting system capacity would be sized to match the maximum flow of 16.4 mgd to secondary treatment.
- Two new 80 foot diameter primary clarifiers would be constructed to handle maximum flow to secondary treatment.
- The internal recycle from solids treatment processes would be routed to the re-aeration zone of the aeration basin.
- The original coarse bubble diffusers were replaced with fine bubble membrane disc diffusers in December 2002.
- An interim change was made in operation of the treatment plant in mid June, 2003. Aeration Basin 4 has been converted from WAS storage to an anoxic zone and the secondary treatment process is operated in the Modified Ludzack Ettinger (MLE) mode for nitrification and denitrification. Aeration Basin 1 is currently being used as WAS storage instead of Basin 4. In the proposed upgrade, Aeration Basin 1 will be divided into a re-aeration basin and anoxic basin. The re-aeration basin is designed to handle the periodic ammonia loading spikes from the solids treatment internal recycle and to maintain the effluent ammonia level within the discharge permit requirement. Basins 2 through 4 will be used as aerobic basins. Between 2020 and build-out, space will be allocated for an additional aeration basin.
- Based on the results of the BIOWIN computer simulations, the maximum capacity of the existing aeration basin, with the Aeration Basin No. 4 back in operation, was estimated to be sufficient to treat the projected 2020 peak day flow of 15.9 mgd and 0.5 mgd of internal recycle flow from solids treatment processes. Flow above this level (16.4 mgd) would be diverted to the HRC system for enhanced primary treatment.
- The new 1.2 million gallon aeration basin would be delayed until after Phase 2 and would be designed at a later date to handle the projected build-out peak day flow.
- Two new 85 foot diameter secondary clarifiers would be installed by 2015. Of the two existing secondary clarifiers, the peripheral feed Secondary Clarifier No. 1 would be removed from service. A total of three (two new and one existing) would be in service by 2015. Space for two more 85 foot diameter secondary clarifiers would be allocated for the build-out scenario to bring the total number of clarifiers in operation to five.

5.0 REVISED MASS BALANCES

Mass balances were prepared based on the results of the BIOWIN computer simulations. The mass balances compared the amount of projected pollutants discharged into the Skagit River by: (a) the CSP-proposed treatment scheme, and (b) the modified treatment scheme proposed in the CSP amendment. The years compared were 2005, 2010, 2015, and 2020. The revised flow projection indicates that re-routing of excess peak flow around primary treatment units and the secondary biological treatment unit would occur under the peak hour flow scenarios in 2015 and 2020. Results of mass balances for these two scenarios for both treatment schemes are presented in Figure 1 and Figure 2.

The following assumptions were used in preparing the mass balances:

- The CSP treatment scheme could treat up to 18.3 mgd of peak hour flow before re-routing is needed. Excess peak flow above 18.3 mgd would be re-routed to a dedicated CSO clarifier for primary treatment. Secondary effluent would be blended with the CSO clarifier effluent before discharge.

- The treatment scheme proposed in the amendment could treat up to 16.4 mgd of peak hour flow before re-routing is needed. Excess peak flow above 16.4 mgd would be re-routed to a dedicated HRC for enhanced primary treatment. Secondary effluent would be blended with the HRC effluent before discharge.

- The primary clarifier is anticipated to have a BOD removal efficiency of 35 percent, a TSS removal efficiency of 55 percent, and an ammonium (NH_4) removal efficiency of 0 percent.

- The CSO clarifier originally proposed in the CSP is anticipated to have the same performance as the primary clarifier (35 percent BOD removal, 55 percent TSS removal, and 0 percent NH4 removal).

- The high rate clarifier is anticipated to have an enhanced BOD removal efficiency of 60 percent, an enhanced TSS removal efficiency of 80 percent, and an NH_4 removal efficiency of 0 percent.

- Both the secondary biological treatment unit in the CSP treatment scheme and the secondary biological treatment unit in the amendment treatment scheme are anticipated to produce an effluent with 10 milligrams per liter (mg/L) of BOD, 15 mg/L of TSS, and 4 mg/L of NH_4, based on BIOWIN computer simulation results.

As shown in Figure 1 and Figure 2, the amendment treatment scheme is anticipated to discharge significantly lower BOD (979 lb/d in 2015 and 1,141 lb/d in 2020) and TSS (1,247 lb/d and 1,442 lb/d in 2020) loadings, and a similar amount (within 30 lb/d difference in 2015 and 2020) of NH_4 loading into the Skagit River as compared with the CSP treatment scheme. The effluent qualities of either treatment scheme would satisfy the anticipated future National Pollutant Discharge Elimination System (NPDES) requirement. However, the amendment treatment scheme would provide a net positive environmental benefit over the CSP treatment scheme based on the results of the mass balance analyses.

Figure 1: Amendment Option

Influent

		2015	2020
Flow	mgd	25.90	25.90
BOD Load	lb/d	16847	19895
BOD Conc	mg/L	78.0	92.1
TSS Load	lb/d	15948	18834
TSS Conc	mg/L	73.8	87.2
NH4 Load	lb/d	1235	1459
NH4 Conc	mg/L	5.7	6.8

Plant Drain Recycle

		2015	2020
Flow	mgd	0.51	0.61
BOD Load	lb/d	233	275
BOD Conc	mg/L	54.3	54.3
TSS Load	lb/d	221	261
TSS Conc	mg/L	51.6	51.6
NH4 Load	lb/d	0	0
NH4 Conc	mg/L	0.0	0.0

Influent + Solids Treatment Recycle

		2015	2020
Flow	mgd	26.41	26.51
BOD Load	lb/d	17080	20170
BOD Conc	mg/L	77.5	91.2
TSS Load	lb/d	16170	19095
TSS Conc	mg/L	73.4	86.4
NH4 Load	lb/d	1235	1459
NH4 Conc	mg/L	5.6	6.6

1° Clarification

		2015	2020
Flow	mgd	16.40	16.40
Inf BOD	lb/d	10605	12479
	mg/L	77.5	91.2
Eff BOD	lb/d	6893	8112
	mg/L	50.4	59.3
Inf TSS	lb/d	10039	11814
	mg/L	73.4	86.4
Eff TSS	lb/d	4518	5316
	mg/L	33.0	38.9
Inf NH4	lb/d	767	903
	mg/L	5.6	6.6
Eff NH4	lb/d	767	903
	mg/L	5.6	6.6
Efficiencies			
BOD	%	35%	35%
TSS	%	55%	55%
NH4	%	0%	0%

ACTIFLO High Rate Clarification

		2015	2020
Flow	mgd	10.01	10.11
Inf BOD	lb/d	6475	7691
	mg/L	77.5	91.2
Eff BOD	lb/d	2590	3076
	mg/L	31.0	36.5
Inf TSS	lb/d	6130	7281
	mg/L	73.4	86.4
Eff TSS	lb/d	1226	1456
	mg/L	14.7	17.3
Inf NH4	lb/d	468	556
	mg/L	5.6	6.6
Eff NH4	lb/d	468	556
	mg/L	5.6	6.6
Efficiencies			
BOD	%	60%	60%
TSS	%	80%	80%
NH4	%	0%	0%
Vs CSP Treated CSO Effluent			
BOD	lb/d	-820	-983
TSS	lb/d	-1009	-1204
NH4	lb/d	34	39

Dewatering Filtrate Recycle

		2015	2020
Flow	mgd	0.00	0.00
BOD Load	lb/d	0	0
BOD Conc	mg/L	NA	NA
TSS Load	lb/d	0	0
TSS Conc	mg/L	NA	NA
NH4 Load	lb/d	180	213
NH4 Conc	mg/L	NA	NA

Reaeration

		2015	2020
Flow	mgd	0.00	0.00
Inf BOD	lb/d	0	0
	mg/L	NA	NA
Eff BOD	lb/d	0	0
	mg/L	NA	NA
Inf TSS	lb/d	0	0
	mg/L	NA	NA
Eff TSS	lb/d	0	0
	mg/L	NA	NA
Inf NH4	lb/d	180	213
	mg/L	NA	NA
Eff NH4	lb/d	18	21
	mg/L	NA	NA
Efficiencies			
BOD	%	NA	NA
TSS	%	NA	NA
NH4	%	90%	90%

Activated Sludge

		2015	2020
Flow	mgd	16.40	16.40
Inf BOD	lb/d	6893	8112
	mg/L	50.4	59.3
Eff BOD	lb/d	1368	1368
	mg/L	10.0	10.0
Inf TSS	lb/d	4518	5316
	mg/L	33.0	38.9
Eff TSS	lb/d	2052	2052
	mg/L	15.0	15.0
Inf NH4	lb/d	785	924
	mg/L	5.7	6.8
Eff NH4	lb/d	547	547
	mg/L	4.0	4.0
Efficiencies			
BOD	%	80%	83%
TSS	%	55%	61%
NH4	%	30%	41%
Applied to Secondary			
BOD	lb/d	6893	8112
TSS	lb/d	4518	5316
NH4	lb/d	947	1115
Removed by Secondary			
BOD	lb/d	5525	6744
TSS	lb/d	2466	3265
NH4	lb/d	400	568
Overall Secondary Efficiencies			
BOD	%	80%	83%
TSS	%	55%	61%
NH4	%	42%	51%
Vs CSP Secondary Effluent			
BOD	lb/d	-158	-158
TSS	lb/d	-238	-238
NH4	lb/d	-63	-63

Blended Effluent

		2015	2020
Flow	mgd	26.41	26.51
BOD Load	lb/d	3958	4444
BOD Conc	mg/L	18.0	20.1
TSS Load	lb/d	3278	3508
TSS Conc	mg/L	14.9	15.9
NH4 Load	lb/d	1015	1103
NH4 Conc	mg/L	4.6	5.0
Total Applied			
BOD	lb/d	17080	20170
TSS	lb/d	16170	19095
NH4	lb/d	1415	1672
Total Removed			
BOD	lb/d	13122	15726
TSS	lb/d	12892	15587
NH4	lb/d	400	568
Total Efficiencies			
BOD	%	77%	78%
TSS	%	80%	82%
NH4	%	28%	34%
Vs CSP Blended Effluent			
BOD	lb/d	-979	-1141
TSS	lb/d	-1247	-1442
NH4	lb/d	-30	-25

Figure 2: CSP Option

Influent		2015	2020
Flow	mgd	25.90	25.90
BOD Load	lb/d	16847	19895
BOD Conc	mg/L	78.0	92.1
TSS Load	lb/d	15948	18834
TSS Conc	mg/L	73.8	87.2
NH4 Load	lb/d	1235	1459
NH4 Conc	mg/L	5.7	6.8

Plant Drain Recycle		2015	2020
Flow	mgd	0.51	0.61
BOD Load	lb/d	233	275
BOD Conc	mg/L	54.3	54.3
TSS Load	lb/d	221	261
TSS Conc	mg/L	51.6	51.6
NH4 Load	lb/d	180	213
NH4 Conc	mg/L	42.0	42.0

Influent + Solids Treatment Recycle		2015	2020
Flow	mgd	26.41	26.51
BOD Load	lb/d	17080	20170
BOD Conc	mg/L	77.5	91.2
TSS Load	lb/d	16170	19095
TSS Conc	mg/L	73.4	86.4
NH4 Load	lb/d	1415	1672
NH4 Conc	mg/L	6.4	7.6

1° Clarification		2015	2020
Flow	mgd	18.30	18.30
Inf BOD	lb/d	11833	13925
	mg/L	77.5	91.2
Eff BOD	lb/d	7692	9051
	mg/L	50.4	59.3
Inf TSS	lb/d	11203	13183
	mg/L	73.4	86.4
Eff TSS	lb/d	5041	5932
	mg/L	33.0	38.9
Inf NH4	lb/d	981	1154
	mg/L	6.4	7.6
Eff NH4	lb/d	981	1154
	mg/L	6.4	7.6
Efficiencies			
BOD	%	35%	35%
TSS	%	55%	55%
NH4	%	0%	0%

CSO Clarification		2015	2020
Flow	mgd	8.11	8.21
Inf BOD	lb/d	5247	6245
	mg/L	77.5	91.2
Eff BOD	lb/d	3410	4059
	mg/L	50.4	59.3
Inf TSS	lb/d	4967	5912
	mg/L	73.4	86.4
Eff TSS	lb/d	2235	2660
	mg/L	33.0	68.9
Inf NH4	lb/d	435	518
	mg/L	6.4	7.6
Eff NH4	lb/d	435	518
	mg/L	6.4	7.6
Efficiencies			
BOD	%	35%	35%
TSS	%	55%	55%
NH4	%	0%	0%
Vs Amendment ACTIFLO Effluent			
BOD	lb/d	820	983
TSS	lb/d	1009	1204
NH4	lb/d	-34	-39

Activated Sludge		2015	2020
Flow	mgd	18.30	18.30
Inf BOD	lb/d	7692	9051
	mg/L	50.4	59.3
Eff BOD	lb/d	1526	1526
	mg/L	10.0	10.0
Inf TSS	lb/d	5041	5932
	mg/L	33.0	38.9
Eff TSS	lb/d	2289	2289
	mg/L	15.0	15.0
Inf NH4	lb/d	981	1154
	mg/L	6.4	7.6
Eff NH4	lb/d	610	610
	mg/L	4.0	4.0
Efficiencies			
BOD	%	80%	83%
TSS	%	55%	61%
NH4	%	38%	47%
Applied to Secondary			
BOD	lb/d	7692	9051
TSS	lb/d	5041	5932
NH4	lb/d	981	1154
Removed by Secondary			
BOD	lb/d	6165	7525
TSS	lb/d	2752	3643
NH4	lb/d	370	544
Overall Secondary Efficiencies			
BOD	%	80%	83%
TSS	%	55%	61%
NH4	%	38%	47%
Vs Amendment Secondary Effluent			
BOD	lb/d	158	158
TSS	lb/d	238	238
NH4	lb/d	63	63

Blended Effluent		2015	2020
Flow	mgd	26.41	26.51
BOD Load	lb/d	4937	5585
BOD Conc	mg/L	22.4	25.3
TSS Load	lb/d	4525	4950
TSS Conc	mg/L	20.5	22.4
NH4 Load	lb/d	1045	1128
NH4 Conc	mg/L	4.7	5.1
Total Applied			
BOD	lb/d	17080	20170
TSS	lb/d	16170	19095
NH4	lb/d	1415	1672
Total Removed			
BOD	lb/d	12143	14585
TSS	lb/d	11645	14145
NH4	lb/d	370	544
Total Efficiencies			
BOD	%	71%	72%
TSS	%	72%	74%
NH4	%	26%	33%
Vs Amendment Blended Effluent			
BOD	lb/d	979	1141
TSS	lb/d	1247	1442
NH4	lb.d	30	25

6.0 CONCLUSIONS

The Comprehensive Sewer Plan Amendment seeks to include an HRC option to treat peak storm flow at the WWTP. The capacity of the existing aeration basin would be maximized to provide peak flow capacity of 16.4 mgd for full secondary treatment, without construction of new aeration basins.

Based on results of the BIOWIN computer simulation using the revised flow projections, it is anticipated that the small reduction (1.9 mgd) in full secondary treatment capacity is more than compensated by the additional HRC capacity of 12 MGD and the enhanced treatment efficiency of the HRC technology over the conventional primary-type CSO clarifier described in the original proposal.

The amendment treatment scheme is anticipated to discharge significantly lower BOD (979 lb/d in 2015 and 1,141 lb/d in 2020) and TSS (1,247 lb/d and 1,442 lb/d in 2020) loadings, and a similar amount (within 30 lb/d difference in 2015 and 2020) of NH_4 loading into the Skagit River as compared with the CSP treatment scheme. The effluent qualities of either treatment scheme would satisfy the anticipated future National Pollutant Discharge Elimination System (NPDES) requirement. However, the amendment treatment scheme would provide a net positive environmental benefit over the CSP treatment scheme based on the results of the mass balance analyses.

The City would also be prudent to continue to monitor the performance of the Central CSO Regulator system through 2010 to provide an additional six years of flow data. The data collected will be used to further refine the CSO compliance alternative and optimize the ultimate execution of the Phase 2 and Phase 3 CSO Reduction Plan.

APPENDIX A

Alternative Peak Flow Treatment

CITY OF MOUNT VERNON, WASHINGTON

WASTEWATER TREAMENT PLANT UPGRADE

PREDESIGN TECHNICAL MEMORANDUM

Prepared by:	Kenneth Hui, PE
Reviewed by:	Dan Olson, PE John Koch, PE
Date:	November 2003
Subject:	High Rate Clarification Pilot Testing Performance Summary
Project Number:	09637-006070

1.0 INTRODUCTION

Full-scale high rate clarification (HRC) systems for treatment of combined sewer overflows (CSOs) or sanitary sewer overflows (SSOs) have been installed in various parts of the country. The two major players in HRC systems are USFilter (Actiflo® system) and Ondeo (DensaDeg® system). A selected installation list of full-scale HRC systems in North America is shown in the appendix.

Five cities or counties in Washington and Oregon have conducted pilot studies of one or both types of HRC systems in the past five years. Summaries of these pilot studies are included in the appendix. Pilot study performances conducted across the United States are also included in the appendix as summary tables.

At present, there is one full-scale HRC system in Bremerton, Washington. It has been in operation since 2002.

Table 1 shows the performance summary of HRC pilot studies conducted in the Pacific Northwest.

Table 1: Performance Summary of HRC Pilot Studies in the Pacific Northwest

Location	HRC Unit	Coagulant	Dosage (mg/L)	Eff TBOD (mg/L)	% Removal	Eff TSS (mg/L)	% Removal	Startup Time to Steady State	Overflow Rate (gpm/sf)
Bremerton	A	Ferric Chloride	15 to 45	47	63%	9	71%	10 min to 5 NTU	55
King County	A	Ferric Chloride	110	48	78%	15	94%	10 to 15 min	60
		PACl	34	51	63%	11	96%		60
		Alum	110	45	74%	11	94%		60
Salem	A	Ferric Chloride	40 to 50	NA	NA	NA	>85%	<20[a]	80
		PACl	20	NA	NA	NA	NA		NA
		Alum	80	NA	NA	NA	85%		30
		ACH	20	NA	NA	NA	90%	<10[a]	100
Tacoma	A	Ferric Chloride	100	112	62%	6	98%	10 to 15 min	60
		PACl	45	31	70%	17	87%		60
Portland	A	Ferric Chloride	58	141	41%	5.2	97%	20 min	20
		Alum	120	210	36%	6	96%		40

Location	HRC Unit	Coagulant	Dosage (mg/L)	Eff TBOD (mg/L)	% Removal	Eff TSS (mg/L)	% Removal	Startup Time to Steady State	Overflow Rate (gpm/sf)
Bremerton	D	Ferric Chloride	60	167	61%	21	85%	75 to 95 min to 5 NTU	20 to 30
King County	D	Ferric Chloride	40	NA	NA	11	96%	20 to 55 min	30
		PACl	40	206[b]	57%	41	81%		30
		Alum	60	250[b]	71%	54	87%		30
Salem	D	Ferric Chloride	40	NA	59%	NA	87%		30 to 40
		PACl	30	NA	59%	NA	87%		30 to 40
		Alum	60	NA	59%	NA	87%		30 to 40
		ACH	15	NA	59%	NA	87%	1.5 to 2 hours	30 to 40

Legend: A = ACTIFLO;D = DensaDeg

Note a: Overflow rate not specified for start up test

Note b: COD Value

APPENDIX:

High Rate Clarification

Selected Installation List and Summaries of Pilot Studies

SUMMARIES OF HIGH RATE CLARIFICATION PILOT STUDIES

EXECUTIVE SUMMARY

- Five cities or counties in the Pacific Northwest have experience in pilot testing high rate clarification (Actiflo® and/or DensaDeg® system) in the past five years. They are: the City of Portland, Oregon; City of Tacoma, Washington; City of Salem, Oregon; City of Bremerton, Washington; and King County, Washington.
- In studies where both technologies were tested, effluent qualities from Actiflo® and DensaDeg® were similar.
- Actiflo® could achieve a similar level of treatment at twice the surface overflow rate of DensaDeg® (60 to 80 gpm/ft^2 for Actiflo® and 30 to 40 gpm/ft^2 for DensaDeg®). The size of a similar-capacity Actiflo® system could be half that of a DensaDeg® system.
- In the King County pilot study, DensaDeg® required less ferric and less alum as coagulant than was required by the Actiflo® system treating similar influent. Polyaluminum chloride (PACl) dosages for both systems were similar.
- In the City of Salem pilot study, DensaDeg® and Actiflo® required similar dosages in all four coagulants tested (ferric, alum, PACl, and aluminum chlorhydrate).
- DensaDeg® utilizes recycled sludge as ballast to accelerate settling. Actiflo® requires micro-sand as ballast to accelerate settling.
- Actiflo® produces a dilute sludge stream. The result of the King County study indicated that the sludge from the Actiflo® pilot plant had a solids concentration of approximately 0.6 percent. DensaDeg® produces a thickened sludge stream. The result of the Bremerton study indicated that the sludge from the DensaDeg® pilot plant had a solids concentration of 2 percent. If sludge from high rate clarification is to be treated separately, Actiflo® sludge may require a gravity thickener before digestion or dewatering. However, if sludge from high rate clarification could be discharged back to the sewer for handling by remote facilities or back to the front end of the primary clarifier for handling by onsite facilities, then the thickening step may not be necessary for Actiflo®. DensaDeg® sludge requires lower degree of thickening.
- Actiflo® has a much shorter startup (both dry and wet) time than DensaDeg®. Results from five pilot studies showed that Actiflo® required 10 to 20 minutes for dry or wet start. Results from three pilot studies showed that DensaDeg® required 55 to 120 minutes for dry start, and 20 to 75 minutes for wet start.
- At present, there is one full-scale Actiflo® installation in the Pacific Northwest in Bremerton, Washington. There is no full-scale DensaDeg® installation in the Pacific Northwest.

FULL SCALE HIGH RATE CLARIFICATION PROCESS
NORTH AMERICAN SELECTED INSTALLATION LIST

Actiflo®

Treatment of Combined or Sanitary Sewer Overflows (CSO or SSO)

- St. Bernard, Louisiana, 10 mgd[1] (2001)
- Lawrence, Kansas, 40 mgd (2002)
- Bremerton, Washington, 10 mgd (2002)
- Fort Smith, Arizona, 31 mgd (2003)

Side Stream Treatment (Wastewater Treatment Application)

- Burlington, Canada, 5.8 mgd (2001)
- Santa Fe, California, 4 mgd (2002)

DensaDeg® (various models)

Primary Treatment

- Laval Station De Lapiniere, Quebec, Canada, 160 mgd (1998)
- Beloeil, Quebec, Canada, 15 mgd (1997)
- Saint-Jean Sur Richelieu, Quebec, Canada, 31 mgd (1996)
- Repentigny, Quebec, Canada, 14 mgd (1996)
- Saint-Eustache, Quebec, Canada, 14 mgd (1991)
- Sherbrooke, Quebec, Canada, 38 mgd (1988 to 1991)
- Puebla Station De Barranca Del Conde, Mexico, 11 mgd (2001)
- Puebla Station De San Francisco, Mexico, 34 mgd (2001)
- Puebla Station D' Atoyac Sur, Mexico, 14 mgd (2001)
- Puebla Station D' Alseseca Sur, Mexico, 23 mgd (2001)
- Breckenridge, Colorado, 2 mgd (1998)

[1]million gallons per day

BREMERTON Actiflo® HIGH RATE CLARIFICATION PROCESS
PILOT TESTING SUMMARY – CDM, FEBRUARY 2000

Pilot Test Information:

Duration:	December 8, 1999 to December 16, 1999
Test Flow:	320 gallons per minute (gpm)
Test Overflow Rate:	55 gallons per minute per square foot (gpm/ft^2)
Polymer Type:	Unknown
Polymer Dosage:	0.5 to 1.0 milligrams per liter (mg/L)
Coagulant Type:	Ferric chloride
Coagulant Dosage:	15 to 45 mg/L
Startup Time to Peak Efficiency:	10 minutes to reach 5 NTU (5 minutes to reach 15 NTU)

Performance:

Parameters	Influent	Effluent	Removal (percent)
Turbidity, Nephelometric Turbidity Unit (NTU)	22	3	86
Total Suspended Solids (TSS), mg/L	31	9[a]	71
Total biochemical oxygen demand (TBOD), mg/L	127	47[b]	63
Soluble biochemical oxygen demand (SBOD), mg/L	70	38	46[b]
Insoluble biochemical oxygen demand (BOD), mg/L	57	9	84
Phosphorus, mg/L	2	0.1	95

[a] Bremerton Primary effluent TSS and BOD at 90 mg/L and 70 mg/L, respectively
[b] Probably due to removal of BOD in colloidal size range

Recommended Preliminary Design Criteria:

Initial Design Overflow Rate:	43 gpm/ft^2
Ultimate Design Overflow Rate:	60 gpm/ft^2

Note: The initial design overflow rate of 43 gpm/ft^2 will allow the unit to be operated at an overflow rate of up to 60 gpm/ft^2 in the future for higher treatment capacity without expanding the facility.

Department of Ecology (DOE) Comments on Pilot Study Report:

- The report needed to address why only one overflow rate of 55 gpm/ft^2 was selected for this pilot test.
- The report needed to justify how the recommended design overflow rate was selected and why the same unit would be expected to achieve the treatment goal at a higher overflow rate.
- The report needed to justify how a one-week short-term study could be valid for full-scale plant design.
- Whole effluent toxicity (WET), ammonia or fecal coliform test data were requested.

BREMERTON DensaDeg® HIGH RATE CLARIFICATION PROCESS
PILOT TESTING SUMMARY – ONDEO, MARCH TO APRIL 2000

Pilot Test Information:

Duration:	March to April 2000
Test Flow:	No Data
Test Overflow Rate:	20 to 30 gpm/ft^2
Sludge Recycle:	3 and 4 percent of influent flow
Polymer Type Tested:	LT 22S LT 27 (Percol 727) Erpac AS 47 Erpac AS 45 Cytec A100 Cytec 1596
Polymer Selected:	Percol 727 (ultra-high molecular weight, anionic, acrylamide polymer from Ciba Giegy)
Polymer Dosage:	2 mg/L
Coagulant Type:	Ferric chloride
Coagulant Dosage:	60 mg/L
Startup Time to Peak Efficiency:	Dry start at 55 minutes to reach 15 NTU (35 minutes to fill and 20 minutes to reach 15 NTU) Dry start at 95 minutes to reach 5 to 10 NTU (35 minutes to fill and 60 minutes to reach 5 to 10 NTU) Wet start at 40 minutes to reach 15 NTU Wet start at 75+ minutes to reach 5 to 10 NTU

Performance:

Parameters	Influent	Effluent	Removal (percent)
Turbidity, NTU	79	9	89
TSS, mg/L	147	21	85
TBOD, mg/L	437	167	61
SBOD, mg/L	127	92	28
Insoluble BOD, mg/L	310	75	76
Phosphorus, mg/L	No Data	No Data	No Data

KING COUNTY Actiflo® HIGH RATE CLARIFICATION PROCESS
PILOT TESTING SUMMARY – HDR, JUNE 2002

Pilot Test Information:

Duration:	August 27, 2001 to October 5, 2001
Test Flow:	310 to 350 gpm
Test Overflow Rate:	53.4 to 60.3 gpm/ft^2
Polymer Type Tested:	M155 anionic dry polymer by CIBA Specialty Chemical
	E700 cationic dry polymer by Polydyne
	AE1125 anionic liquid polymer by BetzDearborn
Polymer Selected:	M155 anionic dry polymer by CIBA Specialty Chemical
Polymer Dosage:	0.75 to 0.95 mg/L
Coagulant Type:	Ferric chloride
	PACl
	Alum
Coagulant Dosage:	60 to 110 mg/L ferric chloride
	17 to 34 mg/L PACl
	60 to 110 mg/L alum
Startup Time to Peak Efficiency:	Dry start at 15 minutes
	Wet start at 10 minutes

Ferric Chloride (110mg/L) Performance (0.95 mg/L Polymer) (60 gpm/ft^2):

Parameters	Influent	Effluent	Removal (percent)
Turbidity, NTU	148	4.9	97
TSS, mg/L	264	15	94
TBOD, mg/L	217	48	78
SBOD, mg/L	78	42	46
Insoluble BOD, mg/L	139	6	96
Total chemical oxygen demand (COD), mg/L	838	260	69
Phosphorus, mg/L	2.64	0.25	91

PACl (34 mg/L) Performance (0.95 mg/L Polymer) (60 gpm/ft^2):

Parameters	Influent	Effluent	Removal (percent)
Turbidity, NTU	166	2.7	98
TSS, mg/L	249	11	96
TBOD, mg/L	136	51	63
SBOD, mg/L	70	42	40
Insoluble BOD, mg/L	66	9	86
Total COD, mg/L	648	180	72
Phosphorus, mg/L	2.58	0.09	97

Alum (110 mg/L) Performance (0.95 mg/L Polymer) (60 gpm/ft^2):

Parameters	Influent	Effluent	Removal (percent)
Turbidity, NTU	123	3.7	97
TSS, mg/L	197	11	94
TBOD, mg/L	174	45	74
SBOD, mg/L	72	38	47
Insoluble BOD, mg/L	102	7	93
Total COD, mg/L	894	262	71
Phosphorus, mg/L	2.94	0.24	92

Recommended Preliminary Design Criteria

Injection Tank Detention Time:	1 min
Maturation Tank Detention Time:	3 min
Overflow Rate:	60 gpm/ft^2

Other Findings

- Estimated sand loss during the pilot study was 250 pounds per million gallons produced. According to the manufacturer, the pilot plant was not operated to minimize sand loss. Under full-scale optimal conditions, sand loss would be in the range of 8 to 12 pounds per million gallons of water treated.
- Sludge concentrations ranged between 3,900 and 8,020 mg/L. The overall average sludge concentration for all testing stages was 6,000 mg/L (0.6 percent). This concentration is considered to be dilute compared to conventional primary sludge, which is typically in the range of 20,000 to 40,000 mg/L (2 to 4 percent).

KING COUNTY DensaDeg® 4D HIGH RATE CLARIFICATION PROCESS PILOT TESTING SUMMARY – HDR, JUNE 2002

Pilot Test Information:

Duration:	October 22, 2001 to February 8, 2002
Test Flow:	86 to 215 gpm
Test Overflow Rate:	20 to 50 gpm/ft^2
Sludge Recycle:	3 and 5 percent of influent flow
Polymer Type Tested:	Magnafloc LT22S cationic high molecular weight dry polyacrylamides
	Nalco IC34 anionic high molecular weight emulsion polymer
Polymer Selected:	Nalco IC34
Polymer Dosage:	1.0 mg/L
Coagulant Type:	Ferric chloride
	PACl
	Alum
Coagulant Dosage:	10 to 60 mg/L ferric chloride
	10 to 60 mg/L PACl
	20 to 60 mg/L Alum
Startup Time to Peak Efficiency:	Dry start 55 minutes
	Wet start 20 minutes

Ferric Chloride (40 mg/L) Performance (1 mg/L Polymer) (30 gpm/ft^2):

Parameters	Influent	Effluent	Removal
Turbidity, NTU	88	4	96%
TSS, mg/L	258	11	96%
TBOD, mg/L	No Data	No Data	No Data
SBOD, mg/L	No Data	No Data	No Data
Insoluble BOD, mg/L	No Data	No Data	No Data
Total COD, mg/L	No Data	No Data	No Data
Phosphorus, mg/L	4.25	0.65	85%

PACl (40 mg/L) Performance (1 mg/L Polymer) (30 gpm/ft^2):

Parameters	Influent	Effluent	Removal
Turbidity, NTU	120	11	91%
TSS, mg/L	219	41	81%
TBOD, mg/L	No Data	No Data	No Data
SBOD, mg/L	No Data	No Data	No Data
Insoluble BOD, mg/L	No Data	No Data	No Data
Total COD, mg/L	484	206	57%
Phosphorus, mg/L	No Data	No Data	No Data

Alum Performance (1 mg/L polymer) (30 gpm/ft^2):

Parameters	Coag. Dose	Influent	Effluent	Removal
Turbidity, NTU	40 mg/L	96	9	90%
TSS, mg/L	60 mg/L	417[a]	54	87%
TBOD, mg/L	No Data	No Data	No Data	No Data
SBOD, mg/L	No Data	No Data	No Data	No Data
Insoluble BOD, mg/L	No Data	No Data	No Data	No Data
Total COD, mg/L	60 mg/L	876	250	71%
Phosphorus, mg/L	40 mg/L	2.66	0.76	71%

[a] Based on test records, during an alum dose of 60 mg/L, the influent turbidity was only 40 NTU. It seems highly unusual to have a TSS of 417 mg/L with a turbidity of only 40 NTU. Also, during pilot testing of Actiflo®, the influent TSS concentration was in the range of 200 to 250 mg/L with corresponding turbidity of 120 to 170 NTU.

Recommended Preliminary Design Criteria:

Reaction Tank Detention Time:	7 minutes
Clarifier Detention Time:	10 minutes
Overflow Rate:	30 gpm/ft^2

Summary of Findings:

- The King County DensaDeg® 4D high rate clarification process pilot test protocol is organized differently from the King County Actiflo® high rate clarification process pilot test protocol. As a result, the pilot study results are presented differently. An attempt was made to summarize the DensaDeg® 4D performance with influent conditions as close as possible to the Actiflo® testing conditions. However, it was impossible to match all the influent conditions.
- In general, based on the recommended preliminary design criteria, the full-scale DensaDeg® 4D system will be two to three times larger in size than the corresponding Actiflo® system with the same treatment capacity.
- The dry start and wet start times for the DensaDeg® 4D system are estimated to be 3.7 times and 2 times higher than the corresponding start times for a similar Actiflo® system. In a side stream wet weather flow treatment scenario, the shorter startup time for the Actiflo® treatment process provides greater flexibility of operations.
- Using ferric chloride ($FeCl_3$) as a coagulant, the DensaDeg® 4D system has similar turbidity and TSS removal performance as the Actiflo® system, but a slightly lower phosphorus removal performance. There is no information on DensaDeg® 4D BOD and COD removal using $FeCl_3$ as coagulant. Optimum test $FeCl_3$ dosage for the DensaDeg® 4D system is approximately one-third of the optimum test dosage for Actiflo® system.
- Using PACl as a coagulant, the DensaDeg® 4D system has a lower (10 to 15 percent lower) performance in turbidity, TSS, and COD removal compared with the Actiflo® system at the **same dosage**. There is no information on DensaDeg® 4D BOD removal using PACl as a coagulant.
- Using alum as a coagulant, the DensaDeg® 4D system has about a 10 percent lower performance in turbidity removal at one-third the alum dosage compared with the Actiflo® system. DensaDeg® 4D TSS removal performance could not be compared accurately due to significantly different influent TSS concentrations. There is no DensaDeg® 4D BOD removal information. The DensaDeg® 4D system has similar COD removal performance as the Actiflo® system at about one-half of the alum dose. The DensaDeg® 4D system has a lower (20 percent lower) performance in phosphorus removal compared with the Actiflo® system at one-third the alum dose.
- Actiflo® produces a more dilute sludge stream than DensaDeg 4D®. There was no direct measurement of sludge information from DensaDeg® 4D during the pilot study.

PILOT TESTING OF Actiflo®, DensaDeg®, AND UV DISINFECTION
CITY OF SALEM, WILLOW LAKE WASTEWATER TREATMENT PLANT
SUMMARY – BRIAN MATSON, CAROLLO ENGINEERS, WEFTEC 2002

Background Information:

- The Willow Lake Wastewater Treatment Plant (WLTP) serves 200,000 people. It has a treatment capacity of 105 mgd. The conveyance system has a capacity of 155 mgd.
- Winter sanitary flow can peak at 300 mgd. The City is required to eliminate SSOs resulting from 5-year storm in winter and 10-year storm in summer by 2010.
- Investigate use of pretreatment, high rate clarification (HRC), and ultraviolet (UV) disinfection at peak excess flow treatment facilities (PEFTFs) remotely at existing SSO sites or at the WLTP.
- Two successive years of wet weather pilot testing at the WLTP for a total of 19 weeks of tests.
- Based on the results, the City is prepared to begin permitting and predesign efforts for a full-scale PEFTF that will treat up to 160 mgd of dilute wastewater at a point where SSOs are currently discharged.

Equipment Tested:

- Actiflo® tested in 2001 and 2002
- DensaDeg® tested in 2002
- Trojan Technologies UV-4000 medium pressure UV system tested in 2001 and 2002
- WEDECO Ideal Horizons Tak55 2-1/143x3 CW low pressure, high output UV system tested in 2002.

HRC Sampling and Analysis:

- Developed BOD:COD ratio allowing COD be used for benchmarking organic removal efficiency
- Collected sufficient BOD analysis to check the variability of BOD:COD ratio

UV Sampling and Analysis:

- Sample collection by plant staff. Split samples were collected for quality control and correlation between methods.
- Analysis of TSS and UV transmittance performed by plant staff and UV vendor personnel.
- *E.Coli* enumeration performed by plant staff using Most Probable Number (MPN) method and UV vendor personnel by membrane filtration method.
- During dose response runs, grab samples collected in triplicate at each UV dose for *E. Coli* enumeration.
- During extended durations of UV runs, automatic lamp sleeve cleaning was disengaged with lamps being wiped clean only at the conclusion of the data collection period.

Influent Dilution:

- Influent dilution using well water was needed for entire six weeks of 2001 testing and at times during 2002 testing due to insufficient rainfall. Dilution water was obtained from a reserve well. Well water provided more alkalinity than the actual dilute influent.
- De-ionized (DI) water was used as dilution water during bench-scale testing and better reflects alkalinity of the actual dilute influent.

Wastewater Characterization:

- Samples of SSO from two overflow locations were collected over five events and compared to the WLTP samples. The results indicated that SSO water is similar to WLTP influent at its most dilute state, with the exception of higher fluoride concentration at the WLTP than at the SSO sites.
- Industrial discharges from several semiconductor manufacturing sources and a latex paint manufacturer caused elevated fluoride and silica concentration, and turbidity spikes of up to 200 NTU. These industrial discharges exerted additional coagulant demand and affected coagulation chemistry.

Coagulants Used and Bench Scale Optimized Dosage:

Coagulant (with 1 mg/L of dry anionic M155 polymer)	Most Dilute Wastewater (BOD and TSS at 40 to 50 mg/L and alkalinity at 50 mg/L)		Least Dilute Wastewater (BOD and TSS at 100 mg/L and alkalinity at 100 mg/L)	
	Range	Optimum	Range	Optimum
Ferric, mg/L	30 – 50	40	40 – 70	60
Alum, mg/L	25 – 35	30	60 – 100	80
Aluminum Chlorhydrate (ACH), mg/L	10 – 15	12	10 – 20	15
Polyaluminum Chloride (PACl), mg/L	20 – 30	25	N/A	N/A

Mitigation of Sand Binding in Actiflo® in Presence of Latex Paint:

- Using alum as a coagulant, microsand ballast became bound together with polymer to form larger gelatinous floc in the clarifier hopper in the presence of latex paint and could no longer be pumped through the hydro-cyclone and returned to the process.
- Sand binding was mitigated by using ferric chloride as a coagulant, or by using ACH or PACl with increased coagulation contact time prior to polymer addition.
- DensaDeg® was not affected by latex paint.

Actiflo® Performance:

Chemical Dosages

- Optimum dose of dry polymers (M725, M155) was 0.5 to 0.6 mg/L for aluminum-based coagulants. Higher doses caused sand binding. Dosage increased to 1 mg/L for ferric.
- Optimum liquid polymer (AE1125) dose was 2.0 mg/L for both iron and aluminum-based coagulants.
- 40 to 50 mg/L of Ferric, 80 mg/L of Alum, 10 to 20 mg/L of ACH, or 20 mg/L of PACl, all of which are within the ranges of bench scale dosages.

Performance at Different Surface Overflow Rate and Coagulant

- When alum was used as a coagulant, TSS removal efficiency was lower than 85 percent if surface overflow rate was higher than 30 gpm/ft^2. Coagulant contact time was not kept constant at higher surface overflow rate.
- When ferric was used as a coagulant, TSS removal efficiency remained higher than 85 percent even if surface overflow rate was 80 gpm/ft^2. Coagulant contact time was between 4 to 8 seconds.
- If coagulant is injected further upstream in influent piping to provide 40+ seconds of contact time, use of ACH as coagulant can achieve close to 90 percent of TSS removal even at a surface overflow rate of 100 gpm/ft^2.
- At a surface overflow rate of 60 gpm/ft^2, with a 5-second ferric contact time or a 43-second ACH contact time, TSS removal efficiency of 85 to 90 percent and BOD removal efficiency of 50 to 70 percent could be achieved.
- Provided sufficient coagulation time, TSS and BOD removal performance similar to that at a surface overflow rate of 60 gpm/ft^2; could be achieved at a higher surface overflow rate of between 80 and 120 gpm/ft^2.

Startup Time

- Less than 20 minutes was required for the unit to complete a dry startup using ferric as a coagulant. Less than 10 minutes was required for the unit to complete a dry startup using ACH as a coagulant.

DensaDeg® Performance:

Chemical Dosages

- Percol 727 polymer was used at a dose of 1.5 mg/L; Nalclear 8173 polymer was used at a dose of 2.0 mg/L.
- 40 mg/L of ferric, 60 mg/L of alum, 15 mg/L of ACH, or 30 mg/L of PACl, all of which are within the ranges of bench scale dosages.
- Sludge recycle at 3.5 and 7 percent of influent flow

Performance at Different Surface Overflow Rate and Coagulant

- Greater than 2 minutes of coagulant contact time was provided for all surface overflow rates tested.
- To achieve 80 to 85 percent TSS removal, surface overflow rates should be limited to 40 gpm/ft^2. Performance deteriorated rapidly at 50 gpm/ft^2 and solids were washed out of the clarifier ultimately leading to a process failure. Performance at 30 gpm/ft^2 was marginally better than that at 40 gpm/ft^2.
- The reactor solids concentration must reach 600 mg/L to achieve acceptable effluent TSS level (around 5 mg/L) if aluminum-based coagulant is used. This solids concentration could be achieved if the surface overflow rate was between 30 and 40 gpm/ft^2.
- A lower reactor solids concentration of 400 mg/L can achieve the same performance if ferric coagulant is used.
- At surface overflow rate of 30 to 40 gpm/ft^2, TSS removal of 87 percent, COD removal of 67 percent, and BOD removal of 59 percent were achieved.
- TSS removal dropped from 78 percent at a surface overflow rate of 50 gpm/ft^2 to 2 percent at 55 gpm/ft^2. Solids started to wash out of the clarifier at this surface overflow rate. After failure occurred, the surface overflow rate was reduced to 40 gpm/ft^2 and the process could not regain performance to pre-failure level even after one hour of operation at the lower surface overflow rate.

Startup Time

- For both ferric and ACH, the dry startup time is between 1.5 and 2 hours at a surface overflow rate of 30 to 40 gpm/ft^2.

UV Disinfection:

Water Quality

	UVT[1] (%)	**TSS (mg/L)**	**Turbidity (NTU)**	***E.coli*** **(per 100 mL) (geometric mean of composite samples taken)**
Screened Raw Influent	40 – 50	40 – 100	70 – 200	1,000,000+
Actiflo® Effluent (using ACH or PACl)	70 – 80	3 – 10	2 – 5	10,000 – 100,000
DensaDeg® Effluent (using ACH or PACl)	70 – 80	6 – 28	4 – 10	10,000 – 100,000

[1]Ultraviolet transmission

Collimated Beam Evaluation of HRC Effluent

- Six runs were conducted. Five runs showed a UV dose of 10 to 40 milli-joules per square centimeter (mJ/cm^2) can reach the target level of less than 126 *E.coli* per 100 mL. One run showed a UV dose of 70 mJ/cm^2 would be required to achieve the target level. It was concluded that a UV dose of 30 to 40 mJ/cm^2 would be sufficient to provide disinfection to HRC effluent.

Pilot Dose Response Curves of HRC Effluent

- The UV dose response curve for Actiflo® effluent with ferric coagulant demonstrated that the disinfection goal of 126 *E.coli* per 100 mL was unattainable even at UV doses greater than 100 mJ/cm^2 due to high absorbance of ferric iron in the carryover floc. Only when UV dose was elevated to approximately 200 mJ/cm^2 did the number of *E.coli* drop below 126 per 100 mL.
- When aluminum-based coagulant was used, the Trojan medium pressure system could meet the geometric mean of 126 per 100 mL in all but three samples at UV doses which ranged from 20 to 50 mJ/cm^2.
- The results of the WEDECO pilot runs indicated that the disinfection goal of 126 *E.coli* per 100 mL was met at a calculated dose of 21 mJ/cm^2.
- The HRC effluent quality corresponded with the water quality of the collimated beam runs with 70 to 84 percent UVT, 3 to 9 mg/L of TSS, and less than 6 NTU.

TACOMA Actiflo® PILOT STUDY SUMMARY
USFilter, FEBRUARY 1999

Pilot Test Information:

Duration:	February 15, 1999 to March 5, 1999
Test Flow:	660 gpm
Test Overflow Rate:	60 gpm/ft^2
Polymer Type:	Allied Colloids 725
Polymer Dosage:	1.5 to 1.6 mg/L
Coagulant Type	Ferric chloride
	PACl
Coagulant Dosage:	70 to 100 mg/L of ferric chloride
	17 to 65 mg/L of PACl
Startup Time to Peak Efficiency:	Wet Start at 10 to 15 minutes

Ferric (100 mg/L) Performance (average 1.25 mg/L Polymer) (60 gpm/ft^2):

Parameters	Influent	Effluent	Removal
Turbidity, NTU	91	6	93%
TSS, mg/L	305	6	98%
TBOD, mg/L	294	112	62%
SBOD, mg/L	No Data	No Data	No Data
Insoluble BOD, mg/L	No Data	No Data	No Data
Total COD, mg/L	689	182	74%
Phosphorus, mg/L	No Data	No Data	No Data

PACl (45 mg/L) Performance (average 1.5 mg/L Polymer) (60 gpm/ft^2):

Parameters	Influent	Effluent	Removal
Turbidity, NTU	67	1.5	98%
TSS, mg/L	135	17	87%
TBOD, mg/L	102	31	70%
SBOD, mg/L	28	25	11%
Insoluble BOD, mg/L	74	6	92%
Total COD, mg/L	260	80	69%
Phosphorus, mg/L	No Data	No Data	No Data

Effect of Surface Overflow Rate on Process Performance (45 mg/L PACl and 1.5 mg/L Polymer):

	$40\ gpm/ft^2$ [(a)] (percent)	$60\ gpm/ft^2$ (percent)	$80\ gpm/ft^2$ [(b)] (percent)
Turbidity Removal	97	97	97
TSS Removal	93	91	90
COD Removal	69	65	70

[a] Influent flow reduced to lower surface overflow rate, thus increasing coagulation time.
[b] Influent flow was kept the same as the 60-gpm/ft^2 trial and surface overflow rate was increased by blocking off 33 percent of the clarifier area, thus keeping the coagulation time the same as the 60-gpm/ft^2 trial.

Summary of Findings

- The performance of Actiflo® with the optimum coagulant and polymer dosage was consistent at surface overflow rates from 40 gpm/ft^2 to 80 gpm/ft^2.
- Sludge solids content from 0.33 to 0.55 percent.
- Over the long term run (nine days of continuous running), the Actiflo® pilot unit's effluent deteriorated during five time periods. These time periods coincided with experimentation involving the use of a streaming current controller (SCC) to automatically adjust coagulant dose. A full-scale peak wet weather flow facility at this time would not be operated with an SCC until long-term experience has proven its reliability. The CSO Actiflo® plant in Colombier, Switzerland operates with a constant coagulant dose that allows the plant to accept influent with fluctuating characteristics and still produce an effluent quality within permitted limits.

Pilot Test Information:

Duration:	December 1997
Test Flow:	No Data
Test Overflow Rate:	20 to 60 gpm/ft^2
Polymer Type:	Allied Colloids LT25
Polymer Dosage:	0.75 mg/L
Coagulant Type:	Ferric chloride Alum
Coagulant Dosage:	50 to 75 mg/L of ferric chloride 120 mg/L of alum
Startup Time to Peak Efficiency:	Wet start at 20 minutes (at 20 gpm/ft^2)

Ferric (58 mg/L) Performance (average 0.75 mg/L Polymer) (20 gpm/ft^2):

Parameters	Influent	Effluent	Removal
Turbidity, NTU	135	5	96%
TSS, mg/L	165	5.2	97%
TBOD, mg/L	238	141	41%
SBOD, mg/L	No Data	No Data	No Data
Insoluble BOD, mg/L	No Data	No Data	No Data
Total COD, mg/L	520	270	48%
Phosphorus, mg/L	5.0	0.1	98%
Copper, micrograms per liter (μg/L)	93	20	78%
Zinc, μg/L	105	58	45%
Lead, μg/L	6	0.8	87%

Alum (120 mg/L) Performance (average 0.75 mg/L Polymer) (40 gpm/ft^2):

Parameters	Influent	Effluent	Removal
Turbidity, NTU	120	12	90%
TSS, mg/L	140	6	96%
TBOD, mg/L	330	210	36%
SBOD, mg/L	No Data	No Data	No Data
Insoluble BOD, mg/L	No Data	No Data	No Data
Total COD, mg/L	600	380	37
Phosphorus, mg/L	4	0.5	91%
Copper, µg/L	45	25	45%
Zinc, µg/L	124	27	78%
Lead, µg/L	5.7	0.4	93%

Summary of Findings:

- Effluent quality was strongly dependent on operating pH, with significantly lower effluent contaminant concentrations at a pH value lower than 6.5 for both ferric and alum coagulants.
- The attempt to complete a mass balance calculation for sludge production was unsuccessful. The sludge flow rate data may be incorrect.
- Limited data suggested that organic material did not accumulate on the sand particles. Conditions in the rapid mix tank and hydro-cyclone may be sufficient to scour the sand.
- There is a strong correlation between the BOD and COD data collected.
- Dosage of ferric chloride was expressed as mg/L of iron (Fe) instead of mg/L of ferric chloride ($FeCl_3$). Also, overflow rate was expressed in gallons per day per square foot (gpd/ft^2) instead of gallons per minute per square foot (gpm/ft^2).

OTHER Actiflo® PILOT STUDY PERFORMANCE IN WASTEWATER / WET WEATHER TREATMENT

Location	Study Date	Overflow Rate (gpm/ft^2)	Coagulant Type	Coagulant Dose (mg/L)	Polymer Type	Polymer Dose (mg/L)	TSS Removal	BOD Removal	COD Removal	Phosphorus Removal
Galveston, TX	3/98	20 – 40	Ferric Chloride	75 – 125	LT25	1.0 – 1.5	80 – 97%	60 – 80%	65 – 90%	No Data
Cincinnati, OH	4/98	30 – 60	Ferric Chloride	20 – 100	Allied Colloids 725	1.0 – 1.5	80 – 90%	40 – 90%	30 – 80%	90 – 99%
Fort Worth, TX	10/98	50 – 80	Ferric Sulphate	77	Allied Colloids 725	1.0	86%	56%	No Data	95%
Jefferson County, AL	10/98	40 – 80	Ferric Chloride	40 – 80	Allied Colloids 725	0.85	85 – 100%	20 – 60%	No Data	No Data
Port Clinton, OH	7/99	20 – 40	Ferric Chloride	75 – 125	Allied Colloids 725	1.0	80 – 97%	40 – 60%	45 – 70%	No Data
Belleville, IL	5/00	60 – 75	Ferric Chloride	30 – 75	Allied Colloids 725	0.75	80 – 95%	60 – 85%	45 – 80%	No Data
Little Rock, AR	8/00	30 – 60	Ferric Chloride	65 – 85	Allied Colloids 725	1.25	86 – 99%	50 – 85%	No Data	No Data
Independence, MO	9/2000	60	Ferric Chloride	40	Ciba Percol 725	0.75 – 1.25	90%	47 – 90%	41 – 90%	No Data

DensaDeg® 4D PILOT STUDY PERFORMANCE IN WASTEWATER / WET WEATHER TREATMENT

Location	Study Date	Overflow Rate (gpm/ft^2)	Coagulant Type	Coagulant Dose (mg/L)	Polymer Type	Polymer Dose (mg/L)	TSS Removal	BOD Removal	COD Removal	Phosphorus Removal
Birmingham, AL	6/98-8/98 (CSO)	29 – 59	Fe	45	Percol 727	1.5	86%	47%	No Data	No Data
Fort Worth, TX	10/98-12/98 (CSO)	30 – 60	Fe	70 – 150	Percol 727	0.75 – 1.75	86%	47%	No Data	No Data
26th Ward, NYC	7/99-8/99 (CSO)	25 – 55	Fe	52 – 63	Percol 727	1.4 – 1.8	70%	58%	No Data	No Data
Halifax, Nova Scotia	11/99-6/00 (1^{O} Inf)	10 – 33	Fe/Al/PACl	18-25 Fe 4-8 Al 8-10 PACl	AS34/ Percol 727	1.0 – 1.2	71%	71%	No Data	No Data
San Francisco, CA	1/00-2/00 (CSO)	30 – 45	Fe	70 – 90	Percol 727	2.0	85%	64%	No Data	No Data
Little Rock, AR	8/00 (1^{O} Inf & Eff)	20 – 40	FE	60	Percol 727	1.5	85 – 89%	72 – 77%	No Data	No Data
Chesterfield, VA	8/02 (1^{O} Eff)	20 – 30	Fe	120	Nalclear 8173	1.5 – 2.5	75%	49%	No Data	No Data

APPENDIX B

November 13, 2003 Washington Department of Ecology Meeting Minutes

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Meeting Notes

Subject:	Peak Flow Treatment Alternatives for the Wastewater Treatment Facilities – DOE Meeting		
Client:	City of Mount Vernon		
Project:	Wastewater Treatment Plant Upgrade - Predesign	Project No:	09637-06070
Meeting Date:	November 13, 2003, 10:00 am - noon	Meeting Location:	Department of Ecology Offices Northwest Regional Office 3190 160th Avenue SE Bellevue, WA
Notes by:	Dan Olson		

Attendees:

Kevin C. Fitzpatrick, DOE Water Quality Program Section Manager
Bernard Jones, DOE Water Quality Program
Mark Henley, DOE Brightwater Facility Engineer
Walt Enquist, City of Mount Vernon – Wastewater Utility Supervisor
John Koch, HDR – Lead Design Engineer
JB Neethling, HDR – Process Design Expert
Dan Olson, HDR - Project Manager

Topics Discussed, Purpose of meeting:

Discuss attached letter to DOE, dated November 12, 2003, regarding improvements to Mount Vernon WWTP. In that letter and in this meeting, we ask for DOE's opinion on "approval of and proceeding with" improvements somewhat different than that included in the Comprehensive Plan. Specifically, we discussed whether Mount Vernon should pursue the treatment facilities improvement with peak flows (CSO) through a high rate clarification process followed by blending with secondary influent and disinfection. This would include dropping the peak flow secondary treatment down from 24 to 16.4 MGD (max, without nitrification) and installing high rate clarification for treatment of peak flows.
As a goal, we wanted DOE to conceptually approve the change in improvement plans and give direction on how to successfully accomplish the proposed revision to the City of Mount Vernon Comprehensive Plan.
To facilitate discussion, we gave a presentation to further support the proposal and to request direction from DOE on next steps in proceeding with the proposed course of action.

Meeting Notes:
Note: The text of the referenced letter is inserted at the end of these minutes. The presentation is attached in hard copy in the project files.

Kevin and Bernard were present at the beginning of the meeting with Mark coming in after about 30 minutes. Mark's arrival gave us an opportunity to go over most of the presentation twice, which appeared to be very helpful.

Kevin Comment at outset: The EPA blending policy is not the WA DOE policy! WA DOE has a team of 6 to 8 engineers working on a blending policy at this time and expect it to be out mid 2004. It will not be as lenient as the EPA proposal.

Dan, John and JB made a presentation to Bernard, Kevin and Mark outlining suggested changes to the improvement plan contained in the Comprehensive Plan. Dan presented slides one and two as introduction and then John and JB continued through the process discussions, site layout change discussions and into the question and answer time. The question and answer session that followed is documented below. The letter and slides are attached as part of the meeting minutes.

Kevin Q: Clarify total flows through secondary.
JB and John A: JB stated 17, clarified by John to be 16.4 max, BOD only and 15.5 nitrified.

Bernard Q: Do you plan on having redundancy with the high rate clarification (HRC)?
John A: Yes. Initially looking at two 6 MGD units, expandable with time to (total through plant) 50 MGD.

Bernard Q: How do pilot study results show the system results under stress?
JB A: Remarkably stable on Russian River. Incoming NTU varies from 2 to 600 and effluent NTU went from 1 to 2. John A: Most sensitive area is chemical injection system.

Kevin Q: Where are you splitting flow from secondary to HRC? How determined?
All A: Take max through plant secondary at 16.4 MGD and over that to HRC. Compare to the calculated 2020 ADMM flow and design plant to handle that or better, rest through HRC. Can take the 2020 ADMM flow without the second aeration basin.

Bernard Comment: DOE may require a long-term control / monitoring plan.

Dan Q: What steps does DOE see to accomplish this for Mount Vernon?
Kevin A: Just do a Comp Plan revision. This is a long-term control measure, and he (DOE) likes to hear about the long-term thinking of CSO planning in conjunction with the secondary treatment facilities.

Kevin Q: How much work has Mount Vernon done in the collection system to reduce CSO's and to separate CSO flows from sanitary?

Walt A, John, Dan A: Walt and John and Dan described the success of the CSO reduction plan. Walt noted the work the City is doing with separating flows where possible and the thinking about possibilities for the future (Second Street Bridge example given). He noted that the City had reduced CSO overflows by 90% with the 60" interceptor, completed in 1999. He also noted that this proposal was aimed at making a similar level of improvement – from 7 per year now to max 1 per year.

Kevin Comment: Kevin noted that he likes to see the separation work going on and noted that this proposal is much better than others they have seen trying to do much less.

Kevin Q: Does Mount Vernon have a Storm Water Utility? Does this Utility have a source of funds to assist in this project?

Walt A: Yes, City does have a Storm Water Utility, formed in 1996. No funds available in this Utility. Walt noted that the Storm Water Utility has been controversial. Kevin and Mark noted that as a normal problem with Storm Water Utilities.

Mark Q: Would your planned secondary capacity for 2020 be equal to the 2020 ADMM flow?

JB and Dan A: Yes, Comp plan projection is 9.9 MGD ADMM and plan is for over 10 MGD, without the second aeration basin, but making modifications described in presentation. JB noted that the design is for a peak flow of close to 17 (incl. recycle).

Kevin Comment: Don't see a problem with going this way. Treating the sanitary flows at all times, treating all combined flows with only one overflow per year. Don't see a problem.

Kevin Q: Have you done an economic analysis of adding more storage as opposed to HRC?

Walt, Dan and John A: Yes. Walt explained that the initial Comprehensive Sewer Combined Sewer Overflow Reduction Plan did an analysis of reduction alternatives, including separation, in-line storage, and treatment alternatives. We also described the interceptor project and costs associated with that project.

Kevin Comment: Here are the general provisions of the current DRAFT blending policy for WA DOE. "You can't hold me to these, as it is still a draft".

1. To allow blending in design, Community may have to complete I/I program including I/I study, meeting standards for inflow (not applicable to combined). The Community cannot use blending as a way around the I/I improvements.
2. There must be a net environmental benefit, a net reduction in discharge to the environment. A net improvement in annual average and maximum month BOD and TSS discharge.
    a. Kevin noted that, because Mount Vernon is treating the CSO flows that are currently happening, the City should be able to easily demonstrate that.
3. Combined Sewer systems are being categorically exempted from the blending policy. They are being treated under the CSO regulations instead.
4. The Blending policy is primarily focused on newer systems that do not include CSO components.
5. With flow blending, assume additional permit requirements and monitoring.
    a. Limits
    b. Study on soluble BOD
    c. Monitoring for parameters – soluble BOD, metals
    d. WET test for toxicity on blended flows – twice annually, quarterly max.
    e. Record keeping on each bypass
6. Must operate secondary process up to 100% before bypass. This is required to assure a net environmental benefit. Blending process puts more responsibilities on Operators.

Mark Q: What assumptions did your analysis use on ballasted sedimentation for mass balance?
JB A: 50% BOD removal and 80% TSS removal
Mark Comment: That's reasonable.

7. Permit limits must be met at all times.
8. pH limits must be met at all times.
9. There will be an annual assessment / report from flow blending – continued demonstration of environmental benefit.
10. DOE is looking at a separate policy for existing combined systems and CSO.

Kevin and Mark Joint Comment: Your actions on this request need to be the Comp Plan amendment request and the public hearing. The draft policy does not come into play.
You would want to demonstrate a net environmental benefit through analysis. Indicate with a mass balance that you would meet you r effluent quality. You should have your inputs and outputs as part of the revision. If secondary capacity is ≥ max month, you are fine. You cannot treat less than max month through secondary. It is not AKART if less. If you are treating max month at 100% and trying to eliminate CSO's, that is good. Mark noted "Good!" Kevin repeated "Good"!

Bernard Q (to Kevin): How will the DOE or EPA policy development impact this request?

Kevin A: Not at all. Just put this through a comp plan revision process, with the associated public hearing. The proposed policies have no bearing on this.

Bernard Comment: This is new technology, so that is your reason for this comp plan revision.

Kevin and Bernard Comment: Can proceed with this direction if you are:

1. Treating maximum month flow through secondary processes.
2. You meet current permit requirements.

Bernard Q: What schedule do you have for Comp Plan approval?

John and Dan A: Have to talk with Walt (time for City to think). Plan on submitting revision in January or late December with approval in First Quarter of 2004.

Bernard Comment: Good. Submit this as new technology.

Mark Q: When was current Comp Plan approved?

Dan A: February 2003. It is a bit embarrassing, but the technology and the regulations were just not quite there when we did the work (most Comp Plan work complete by mid 2002). Not comfortable at the time with this kind of proposal.

Kevin and Bernard Comment: No need to for embarrassment. New technology and receptivity by regulators is just coming on – note policy work just drafted.

John Q: Do we have a go ahead on this approach?

Bernard and Kevin Comment: Go with this. See no problem with conditional approval of ballasted sedimentation. Watch plant performance for 18 months and then follow with final approval. Note that the plant will not experience anything close to design flows in that 18 months, so may want to force it some way to test.

Mark Comment: You should think about separate disinfection of the secondary and ballasted sedimentation flow streams. UV not too effective for ballasted sedimentation.

Kevin Comment: You should look at the security component, as that is very applicable for wastewater facilities. The legislation will come out soon for that. Especially for the chlorine equipment if you go that way.

Dan Comment: We have thought about the UV issue with HRC and will be discussing that with the City, now that we have conceptual approval to go with HRC.

Kevin Comment: Suggest you have Storm Water Utility ratepayers pick up some of the cost for this improvement – but then they are the same folks.

Dan Q: Can you suggest some funding opportunities for the City to fund part of this project, especially the CSO component?

Bernard A: The City WWTP was funded before, so not likely to get funds again. If UV is included in new plant, funds maybe available. Submit an application for items considered new.

Some financial opportunities were discussed – Mount Vernon being a phase II storm water community. May have funds there.

The meeting ended at 11:45 with Bernard noting that he would expect our request for the Comp Plan amendment in the near future. He sees no problem with approval of that amendment in the first quarter of 2004.

Action/Notes:

1. Now that we have conceptual agreement by DOE for using high rate clarification for peak flows (above Average Day, Maximum Month), Walt and the City of Mount Vernon to confirm internally that integrating the CSO component of the Comprehensive Plan into the WWTP project is the way they want to go. This will be the first step, leading to the others below.
2. Upon approval by City, Dan to confirm intentions with letter to DOE from Walt. Dan will meet with Bernard to go over draft of confirmation letter before finalizing letter and sending to Walt for use.
3. Dan will present cost impacts to the City in combining the two projects. This will help in item 4 below. The construction cost impacts include - not building certain parts of the Secondary system, building the high rate clarification treatment and cost savings of integrating components of the two systems (pumping, disinfection).
4. City and HDR to agree on supplement to existing contract to include scope CSO project work.
5. Following 4 above, HDR to pursue Comprehensive Comp Plan amendment with DOE
6. Proceed with rest of contract as shown, leaving the second aeration basin out of the plan.

TEXT OF LETTER TO DOE - November 12, 2003

Mr. Bernard Jones, P.E.
Department of Ecology Northwest Regional Office
3190 160th Avenue SE
Bellevue WA 98008-5452

Re: City of Mount Vernon Comprehensive Plan Update

Dear Mr. Jones,

Thank you for agreeing to meet with us on November 13, 2003 to discuss proposed improvements to the City's Wastewater Treatment Plant. We appreciate the opportunity to present our recent thinking on what improvements to make in our current upgrade and why we should change somewhat from what is shown in our Comprehensive Plan.

The City of Mount Vernon (City) puts a high priority on water quality in the Skagit River as well as on effective use of ratepayer's dollars. To achieve those goals, we believe a modification to the Comprehensive Plan is warranted for the reasons outlined below.

1. The City has one of nine CSO systems in the State and, as such, the flows in our system are highly influenced by rainfall events. The City's system has recorded flows in excess of 35 MGD when the flow to the wastewater treatment plant, Park Street and Division Street overflows are totalized. These peak flows are usually only a few hours in duration. We have carefully analyzed the quality of the flows in our system, including the CSO flows, for the past ten years. Water quality data from these high flow events indicate that the BOD and suspended solids components are very dilute, usually less than 30 mg/l. The high flows, coupled with the low BOD and SS, can interfere with our secondary treatment processes.

2. Both flows and loadings in the City system have increased at a lower rate since 2000, when compared to the years prior to 2000. This is due to changes in growth patterns, collection system improvements to reduce inflow and infiltration, and better pretreatment of industrial wastes by Draper Valley Farms. Average dry weather flow for 2003 is 2.67 MGD and the projected average dry weather flow for 2020 is 4.8 MGD based on the growth patterns and flow data since 1999.

3. Regulators are thinking progressively about the benefits of blending in certain limited situations, as evidenced by EPA's recent draft policy and six principles on blending. I commend you and your co-workers at DOE and EPA on looking seriously at blending as a viable option for treatment facilities such as Mount Vernon's. This draft policy recognizes the need for some flexibility in regulation to achieve the best overall treatment result for the environment. We note that several Western Washington treatment plants are considering this method of achieving water quality goals. For your information, we have attached a brief summary of pilot study results from plants at Bremerton, King County, Tacoma, Salem and Portland, Oregon.

4. The City is working aggressively toward reducing untreated combined sewer overflows to the Skagit River to one event or less per year. As evidence, the number of untreated CSO events to the Skagit River has been reduced from 90 per year to less than 10 per year since the beginning of 1999. This reduction in untreated CSO events is a direct result of the City's commitment to maintain water quality in the Skagit River. In December of 1998 the City completed construction of a 60-inch CSO storage interceptor that allowed the City to make that 90% reduction in overflow events. The WWTP improvement upgrade plan we will present in our meeting and over the next several weeks should allow us to make another 90% reduction in overflows, bringing the City into compliance with the consent order, perhaps earlier than the stipulated 2015 timeframe.

As you know, the City's current Comprehensive Plan contemplated a complete secondary plant that is double the current capacity, both at average day, max month and at peak hour (24 MGD). Given the reasons above, and with your early support of this direction, the City will begin a process to modify the comprehensive plan.

In short, the City proposes to increase the current plant influent capacity of nitrified maximum month flow to 10.0 MGD during the summer months with a peak day nitrified capacity of 15.5 MGD. These flows are above the stated 2020 flows in the Comprehensive Plan of 9.9 maximum month and 13.9 peak day, but are lower than the planned plant capacity of 24 MGD. We would replace the remaining capacity with a high rate clarification process that is more suitable and more cost effective for the dilute flows experienced during storm events. We believe this will meet our permit requirements, satisfy the six principles listed in EPA's draft blending policy and provide additional benefits as described below.

What this means as far as changes to the Comprehensive Plan is that we propose not building the second aeration basin, instead replacing it with a high rate clarification process (i.e. Actiflo or Densadeg). We will continue, as planned, with the new headworks, primary clarifiers, secondary clarifiers, support buildings, site work and pump stations. We also propose implementing seasonal nitrification and the solids processing improvements contemplated in the Comprehensive Plan. One other impact of this change will be a review of the planned UV disinfection to determine its effectiveness with the high rate clarification effluent.

To support this proposal, HDR has completed plant process modeling and capacity analyses. Using a dynamic simulation model, Biowin, the flows and loads to the treatment plant were analyzed with the ultimate goal of determining the capacity of the existing secondary system while maintaining effluent ammonia of less than 5 mg/l at a wastewater temperature of 10 Degrees C. With three secondary clarifiers and two primary clarifiers (as planned in the Comprehensive Plan), the maximum month plant capacity, including internal recycle is 10.0 MGD with a peak day capacity of 15.5 MGD. This configuration will yield a plant effluent in the range of 10 mg/l BOD and SS and 5 mg/l maximum day (<5 mg/l average) ammonia. We propose that the flows above 13.5 MGD be screened, de-gritted and treated with enhanced sedimentation. The enhanced sedimentation process will typically provide SS reduction in the range of 80 to 90% and BOD reduction in the range of 50 to 70%. Our water quality analyses over the past ten years show typical influent BOD and SS concentrations during flows above 13.5 MGD to be less than 35 mg/l and 30 mg/l respectfully. With a blended effluent and anticipated BOD and SS reduction through enhanced sedimentation, the combined total effluent from the treatment facility will be less than 30 mg/l. Prior to discharge to the Skagit River, the combined effluent will be disinfected.

We believe there are many benefits to this proposal. All of them support our two primary goals of high water quality in the Skagit River, including meeting all of our permit requirements, and effective use of ratepayer dollars. In summary, they are:

- The City can incorporate CSO planning into the current project and plan to increase the delivery of wet weather flows to the treatment facility. This will allow the City to treat a higher overall volume of wastewater.
- The WWTP can meet permit requirements with a higher overall flow than the plan outlined in the Comprehensive Plan.
- This approach provides better protection for the biological treatment units from peak flows experienced during storm events.
- This approach provides for more effective use of ratepayer funds.
- The City has the opportunity to meet their consent decree on CSO flows earlier than the 2015 requirement. Should we decide to do that, it would eliminate discharge of raw sewage during those overflow events.

We believe that the proposed process scenario described above will provide an overall high quality total effluent to the Skagit River, provide the maximum level of treatment at the lowest cost and will make the most effective use of dollars paid by the ratepayers of the City of Mount Vernon.

We look forward to our meeting of November 13, 2003 to discuss this proposal in more detail.

Sincerely,

Walt Enquist, Wastewater Utility Supervisor

January 28, 2004
City Comprehensive Plan Amendment Application
Wastewater Utility Element
Project Narrative

The City of Mount Vernon adopted a Comprehensive Sewer Plan (CSP) February 2003. This proposed amendment addresses changes to the CSP long range needs and expected costs for wastewater treatment plant (WWTP) improvements, sewer repair, sewer replacement, and sewer extensions city wide and in the unincorporated urban growth areas.

The City has initiated pre-design work to further refine work included in the CSP. The CSP includes plans to upgrade and expand the wastewater treatment plant WWTP by 2008. The pre-design report will be incorporated into the CSP and the City Comprehensive Plan. The pre-design work includes upgrading the WWTP, Combined Sewer System, and increasing plant organic and hydraulic capacity. The pre-design report includes a recommendation to include High Rate Clarification (HRC) of wet weather flows. HRC of wet weather flows is presented as a means of complying with an agreement with DOE to reduce Combined Sewer Overflows. This agreement requires Mount Vernon to reduce overflow events to an average of one per year no later than January 1, 2015.

The size of the WWTP is approximately 17 acres. Recommendations to expand the WWTP site, if any, will be identified in the pre-design report. Zoning changes at the WWTP, if any, will be public or commercial/limited industrial which are consistent with current WWTP zoning. Impacts, if any, will be addressed through the SEPA process.

The amendment application is consistent with the stated community vision to "expand the economy to support growth, but not compromise the surrounding environment".

The recommendations and costs identified in the CSP will be incorporated into the City Capital Improvement Plan (CIP).

Expansion of the WWTP and sewer infrastructure is essential to the health, safety, and welfare of the City and areas of the lower Skagit River Basin. The City is required through state and federal rules to provide wastewater services to assure water quality regulations are met as the City grows. The proposed amendment is consistent with the goals of Mount Vernon and Skagit County to maintain the quality of the Skagit River and surrounding environment. This amendment enables the City to expand in accordance with density zonings thereby reducing potential detriment to adjacent property owners that can be caused by failing on-site sewage systems. The amendment generally enhances the City's ability to manage City development in accordance with City and community goals.

APPENDIX C

January 30, 2004 Public Hearing Notice

CITY OF MOUNT VERNON
NOTICE OF APPLICATION FOR COMPREHENSIVE PLAN AMENDMENTS

The City of Mount Vernon is accepting applications for 2004 comprehensive plan amendments. Applications for 2004 amendments must be filed with the City of Mount Vernon Development Services Department by 4:00 p.m., January 30, 2004. Applications must be complete in order to be accepted. Application forms, requirements and procedures are available at the City of Mount Vernon Development Services Department, located at 910 Cleveland Avenue, Mount Vernon, WA 98273. For further information, please call Gloria Rivera or Jenefer Creamer at (360) 336-6214.

Published December 16, 2003 and January 07, 2004.

CERTIFICATION PAGE

For

City of Mount Vernon
Comprehensive Sewer Plan Update
#09637-005-002

The engineering material and data contained in this Comprehensive Sewer Plan were prepared under the supervision and direction of the undersigned, whose seal as registered professional engineers are affixed below.

JAMES C. PETERSON
STATE OF WASHINGTON
27574
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 8/20/04

James C. Peterson,
Supervising Engineer

Richard D. Olson,
Project Manager

City of Mount Vernon
Comprehensive Sewer Plan Update
February 2003
Final

Prepared for:

City of Mount Vernon

Prepared by:

HDR Engineering, Inc.

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:

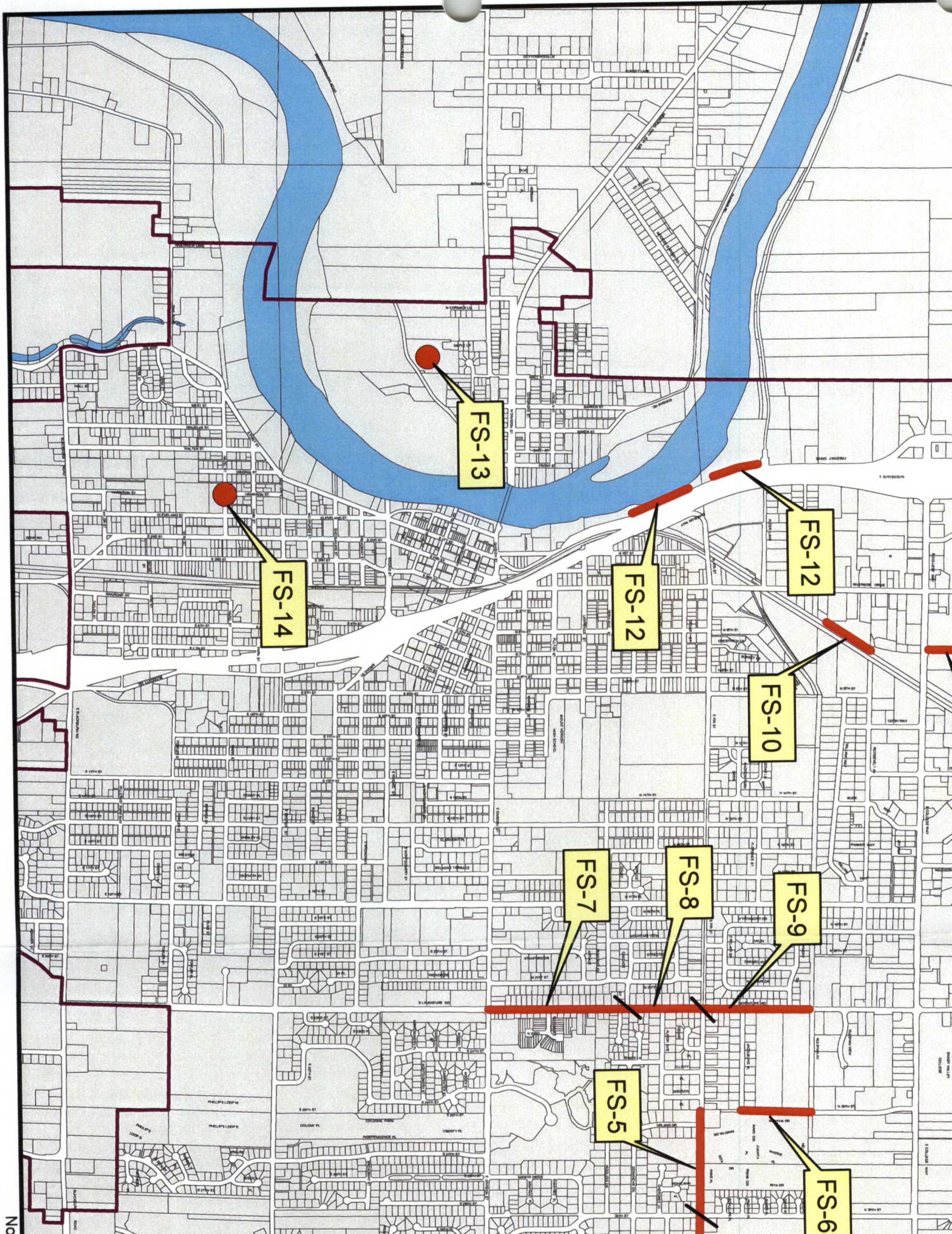
- Convert the Aeration Basin No. 4 to an Activated Sludge Aeration Basin,
- Construct additional Aeration Basins,
- Convert Secondary Clarifier No. 1 to an Aerobic Digester,
- Construct an additional Secondary Clarifier,
- Construct an additional Dissolved Air Floatation Thickener,
- Construct an additional 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.


FS-13
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FS-6

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 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	--

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.

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ABBREVIATIONS

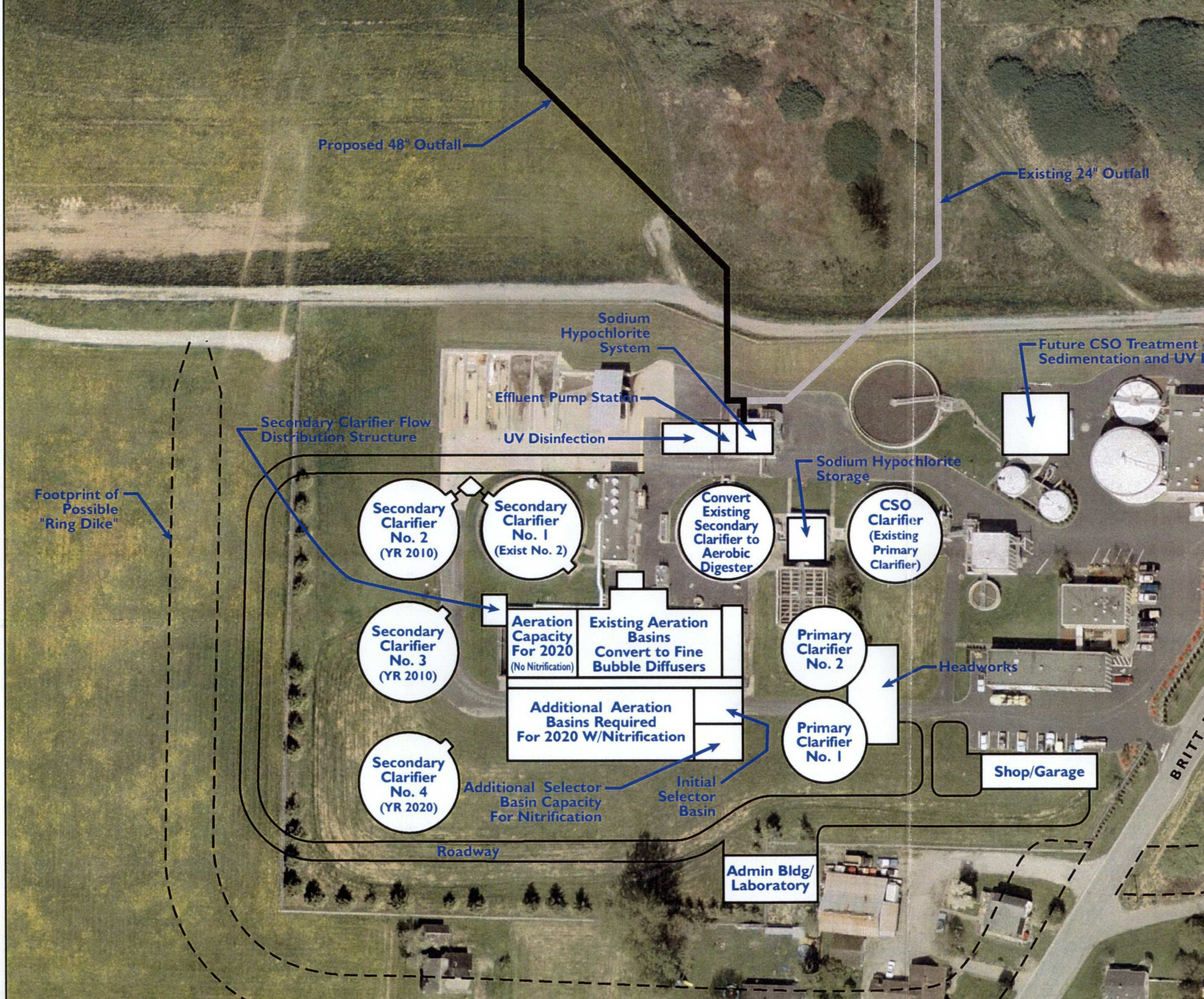
ac	Acre
ADMM	Average Day Maximum Month
AKART	All known, available, and reasonable methods of treatment
BAT	Best Available Technology Economically Achievable
BE/BA	Biological Evaluation/Biological Assessment
BOD	Biochemical Oxygen Demand
BPT	Best Practical Control Technology Currently Available
$CBOD_5$	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



Proposed 48" Outfall
Existing 24" Outfall
Sodium Hypochlorite System
Future CSO Treatment
Sedimentation and UV
Effluent Pump Station
Secondary Clarifier Flow Distribution Structure
UV Disinfection
Sodium Hypochlorite Storage
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
CSO Clarifier (Existing Primary Clarifier)
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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
Roadway
Admin Bldg/ Laboratory
BRITT

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 to 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.

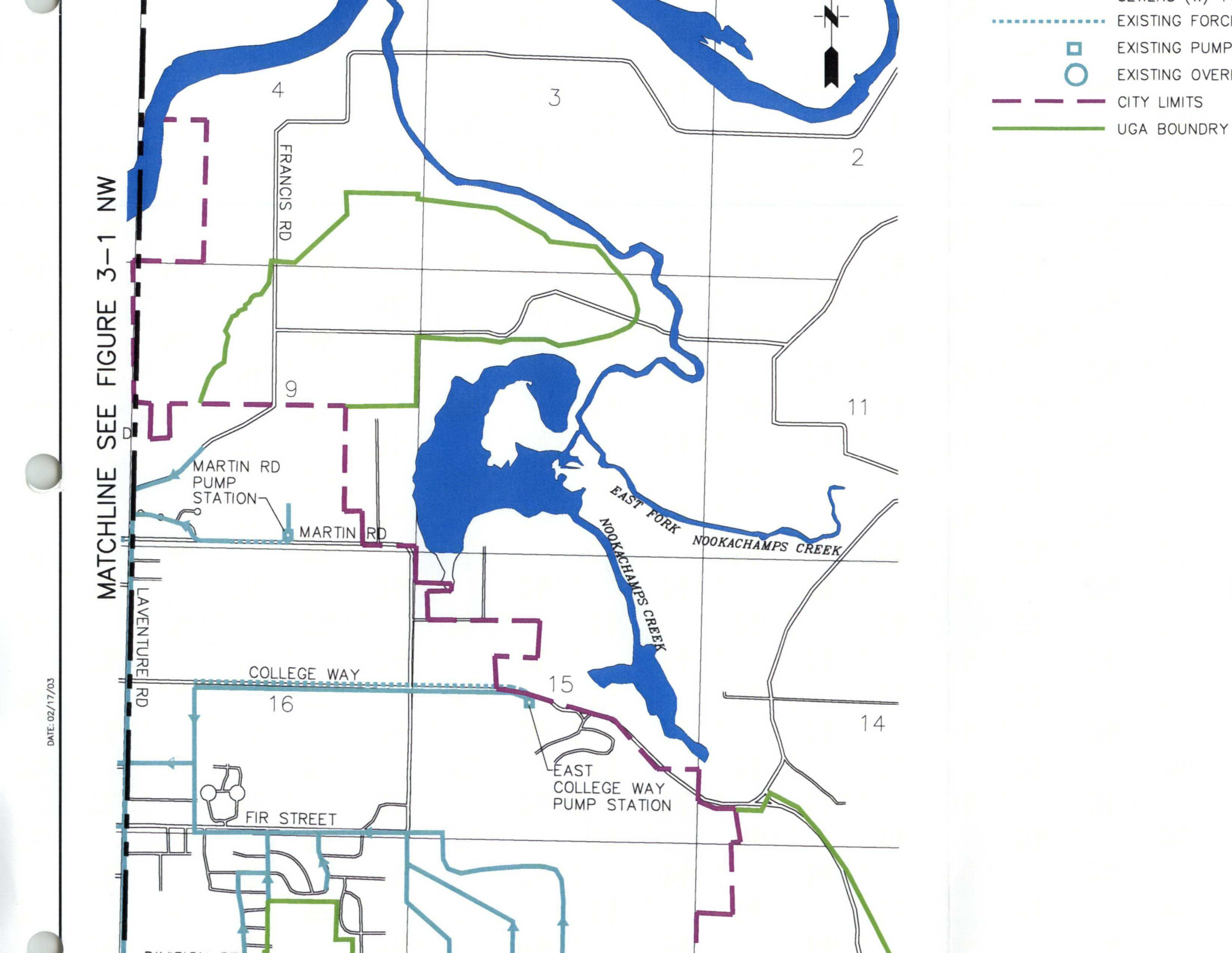
Existing Sewer Service Area

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.

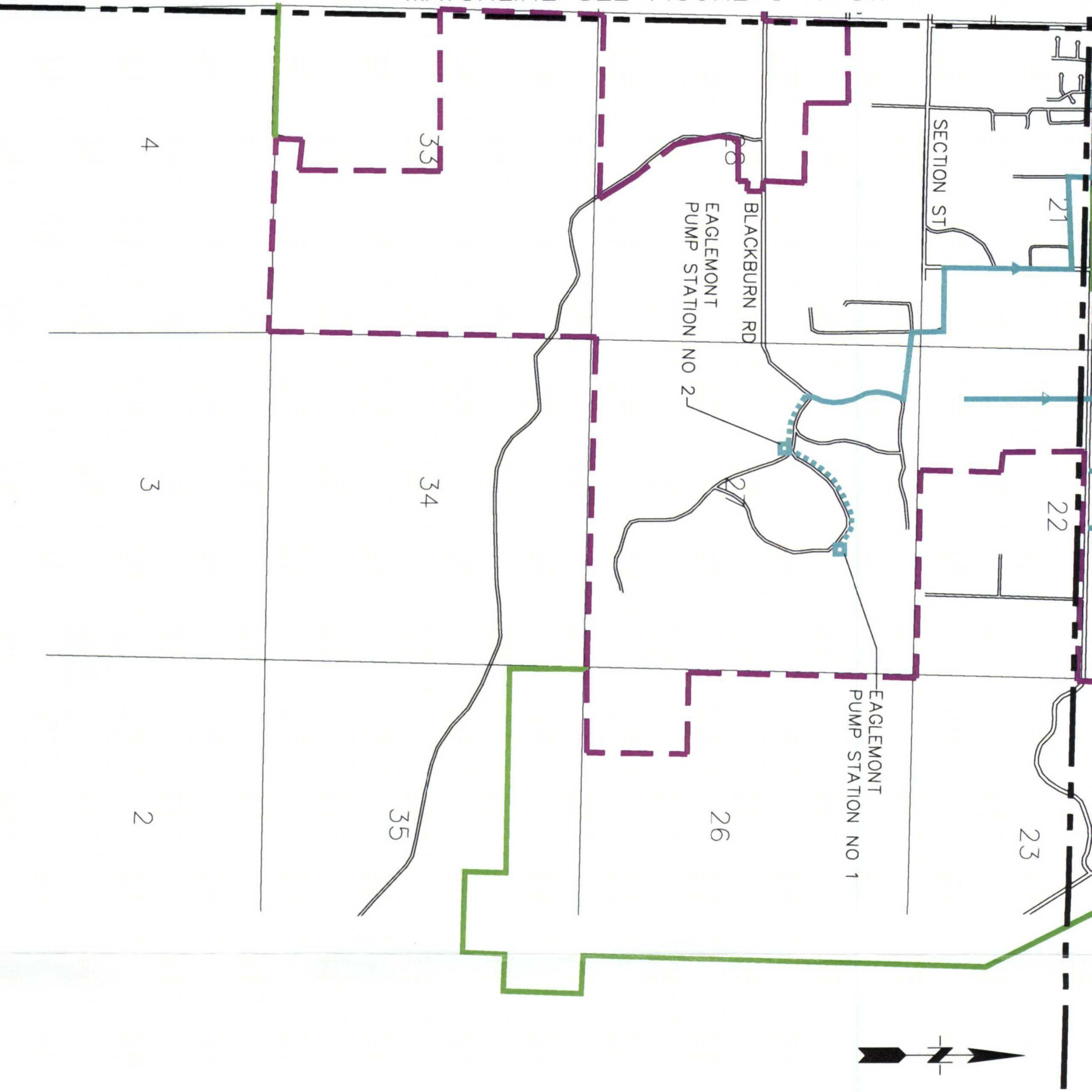
The future sewer service area is the UGA boundary identified by the Skagit County Comprehensive Plan and is delineated graphically in Figure 3-2.



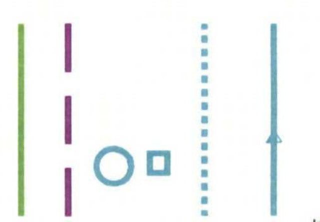
DATE: 02/17/03
MATCHLINE SEE FIGURE 3-1 NW
FRANCIS RD
MARTIN RD PUMP STATION
MARTIN RD
LAVENTURE RD
COLLEGE WAY
FIR STREET
EAST COLLEGE WAY PUMP STATION
EAST FORK NOOKACHAMPS CREEK
NOOKACHAMPS CREEK
4
3
2
9
11
15
16
14
EXISTING FORC
EXISTING PUMP
EXISTING OVER
CITY LIMITS
UGA BOUNDRY

DATE: 02/12/03

MATCHLINE SEE FIGURE 3-1 SW



MATCHLINE SEE FIGURE 3-1 NE



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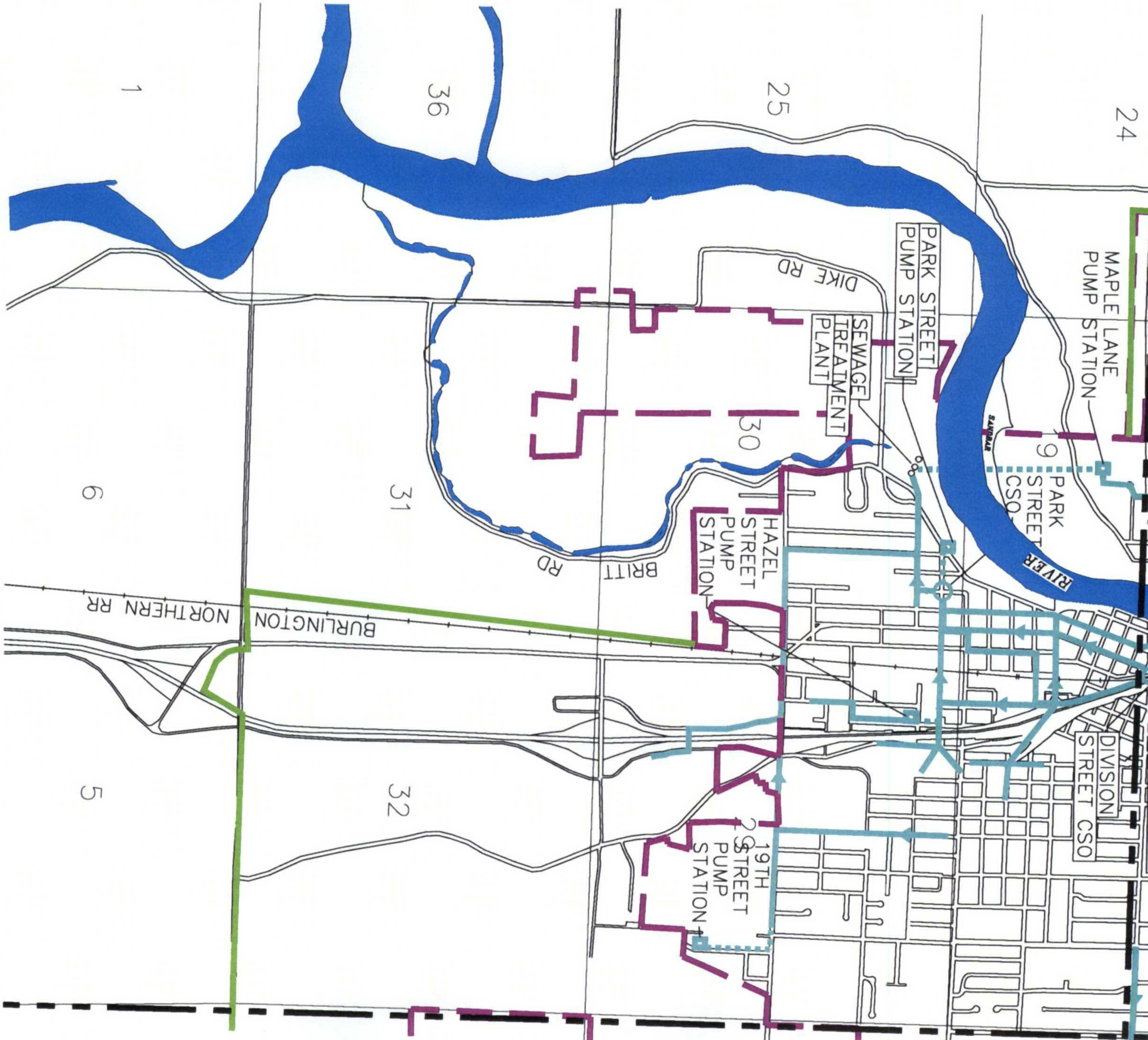
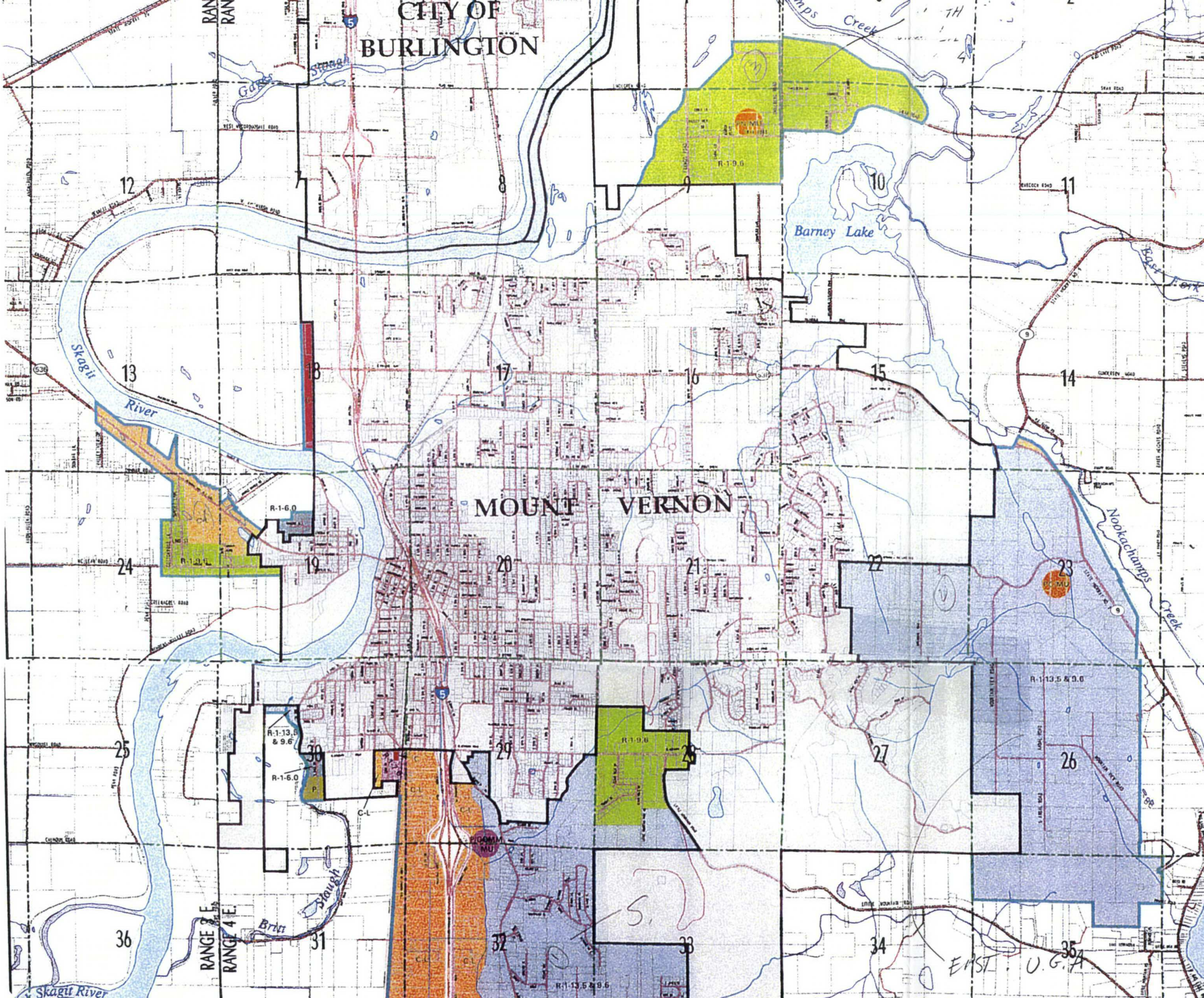


MATCHLINE SEE FIGURE 5–1 SE



CITY OF BURLINGTON
MOUNT VERNON
Skagit River
Barney Lake
Nookachamps Creek
Britt Slough
Gages Slough
RANGE 3 E.
RANGE 4 E.
R-1-9.6
R-1-6.0
R-1-13.5 & 9.6
C-L
P
EAST U.G.A

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.

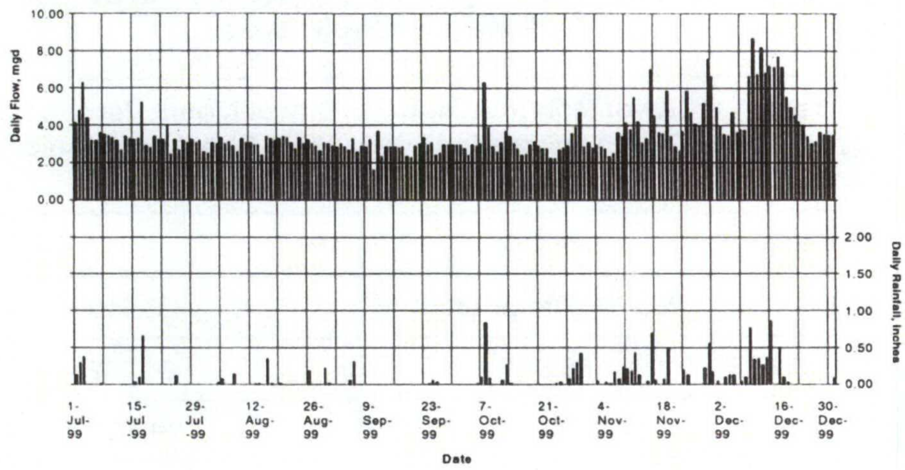
Historical Flows and Loads

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

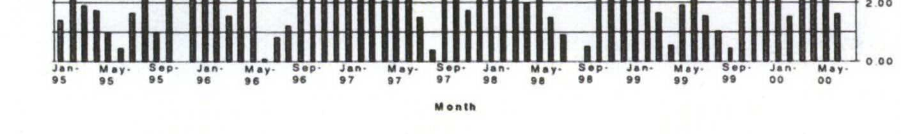
December 31, 1999. This plot illustrates that during late summer the flows reach a base rate of about 2.6 mgd. The plot also illustrates in the dry weather period the five day work week of Draper Valley Farms(DVF), Inc, which discharges from 0.4 to 0.6 mgd when in operation. In November and December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

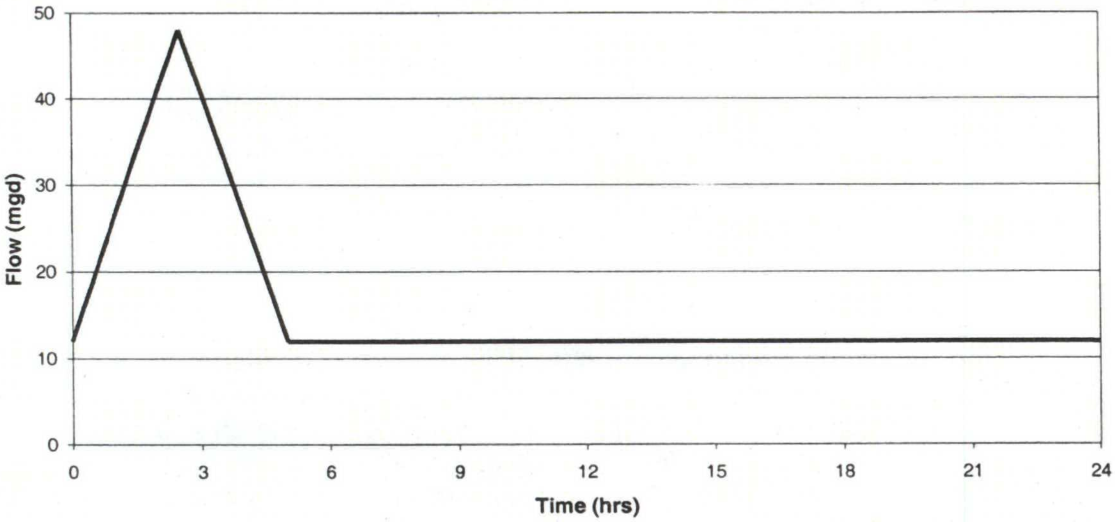
The CSO Baseline for Mount Vernon was established in 1988. It predicted an annual CSO volume of 116.5 MG for the average annual rainfall of 31.95 inches. Based on the 1988 collection system there was a 95 percent confidence that the volume, with an average annual rainfall, would be between 92 MG and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

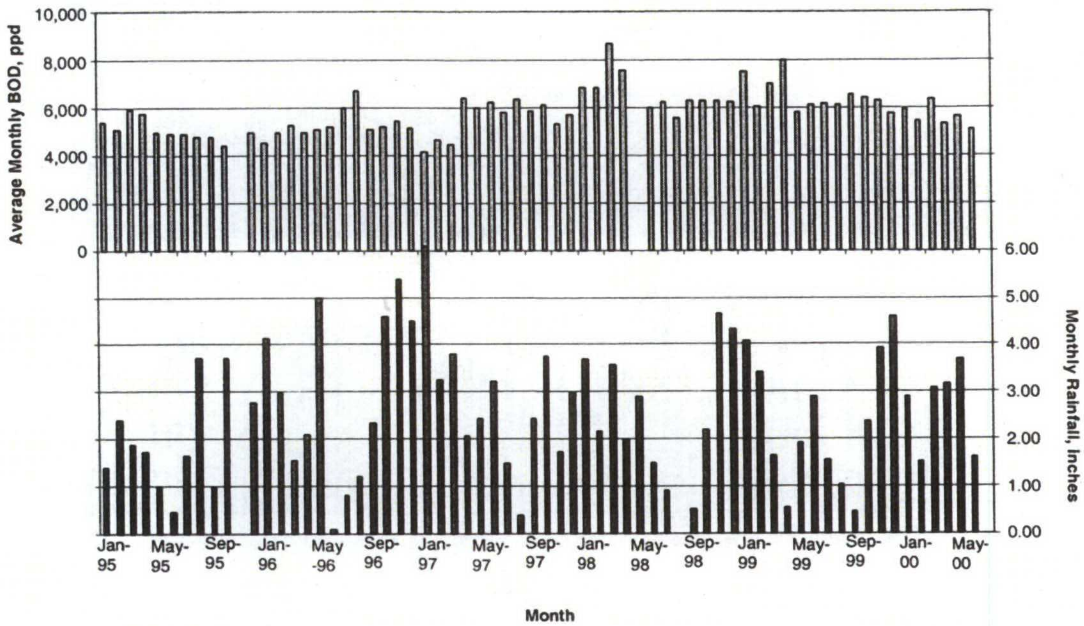
1. Storm flow component estimated from May 16, 1988, storm event.
2. Sanitary and storm components combined flow estimates

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.

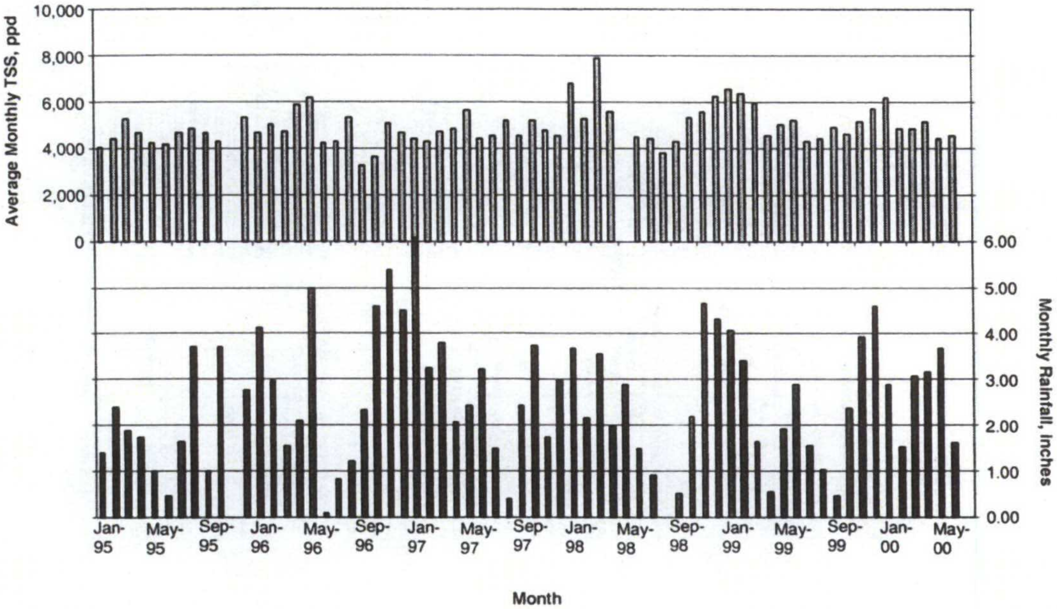
The only current industrial customer, Draper Valley Farms, Inc. (DVF), discharges between 500 and 1,200 ppd of BOD per month. Additional information on these loads is provided in Chapter 6. Based on previous City discussions with DVF it is assumed that the future flows from the plant could approach 0.75 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

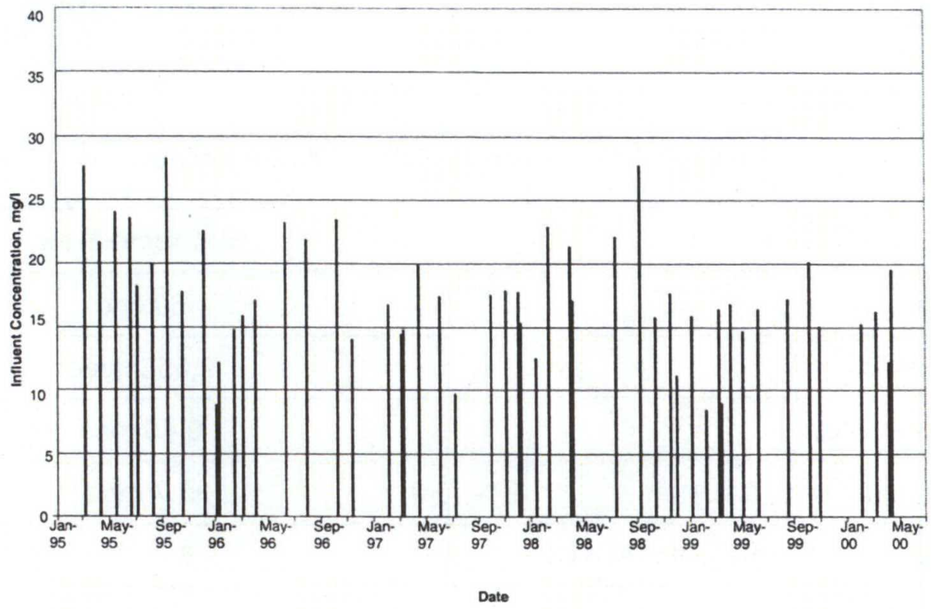
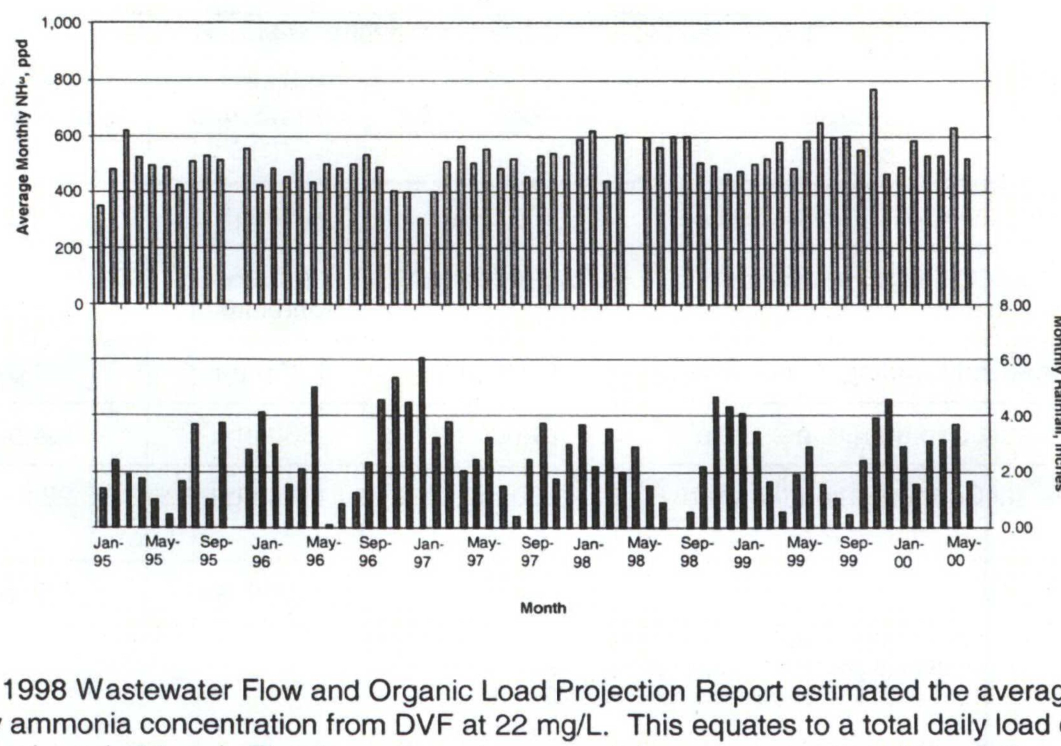


Figure 3-9 City of Mount Vernon Monthly Ammonia Loading



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 mgd[2.]	18.3 mgd[3]
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.

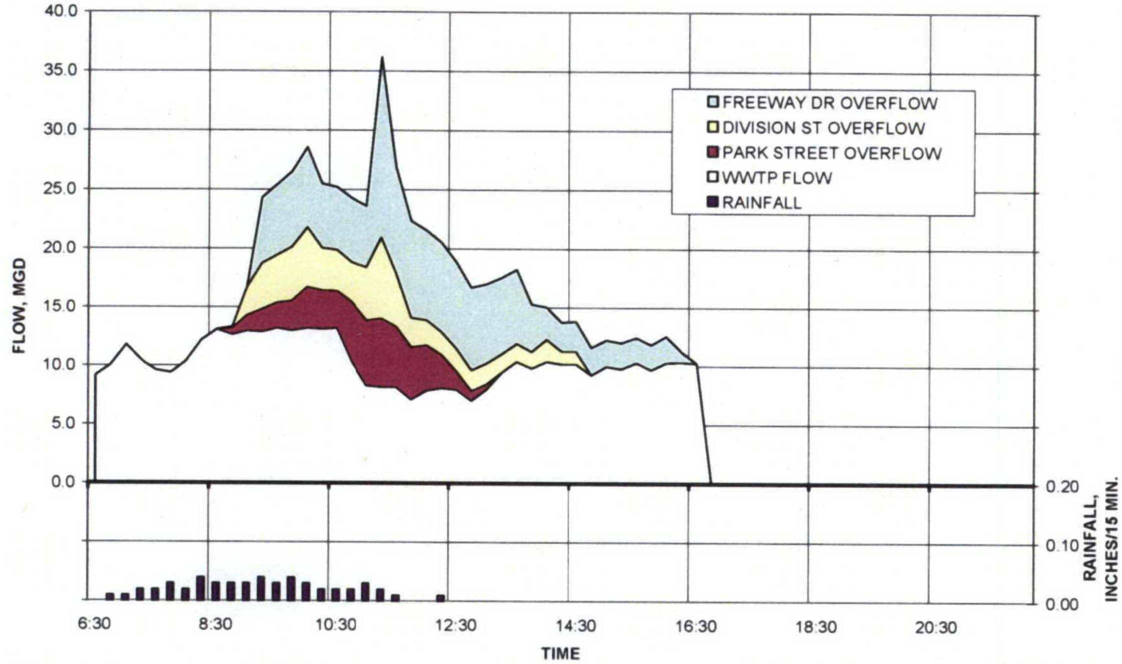
Table 4-2

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



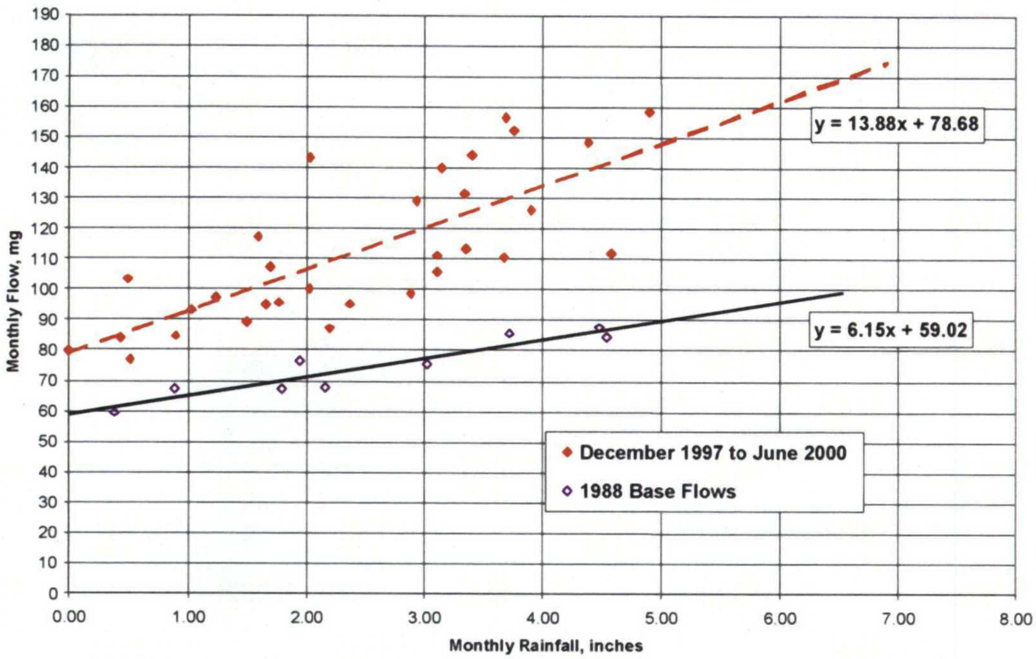
The October 1995 Wastewater Treatment Plant Evaluation evaluated the facilities required based on a peak design system flow rate of 48 mgd. The peak system flow rate observed since the Central CSO Regulator has been in service was 36.2 mgd. A detailed evaluation of the return frequency of this flow rate 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

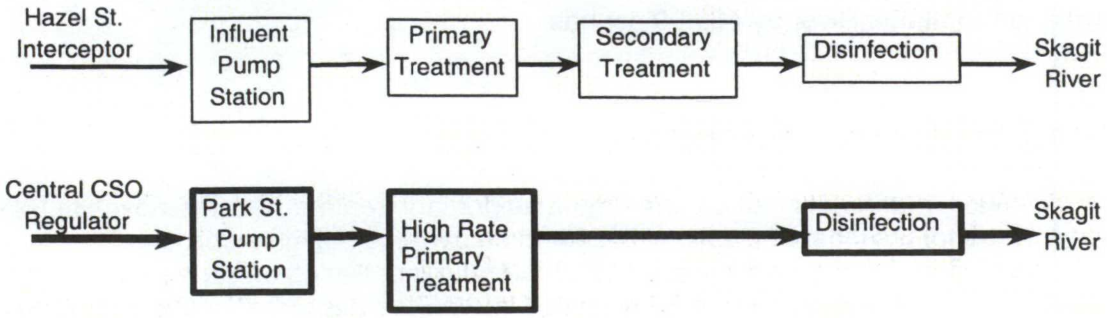
The State of Washington defines CSO treatment as primary treatment that removes at least 50 percent of the total suspended solids (TSS) and an average settlable solids concentration of 0.3 mL/L/hr, with a maximum of 1.9 mL/L/hr. Based on recent CSO treatment projects, the Department of Ecology has interpreted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



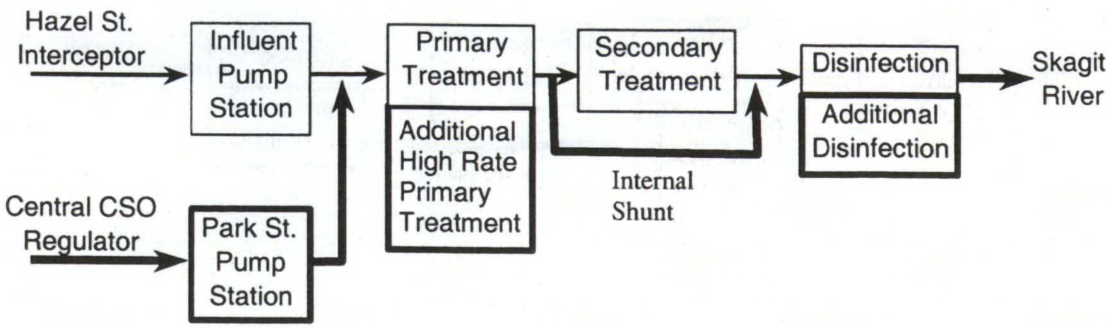
The improvements required for Alternative 1 include:

- 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



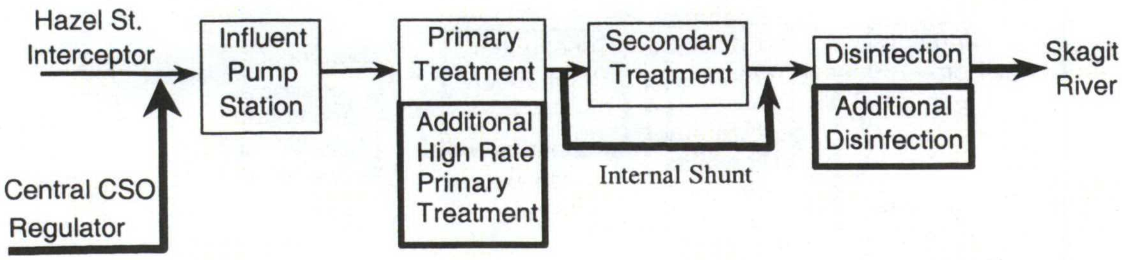
The improvements required for Alternative 2 include:

- 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: 30 mg/L BOD and TSS	NPDES Permit Limits: 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.

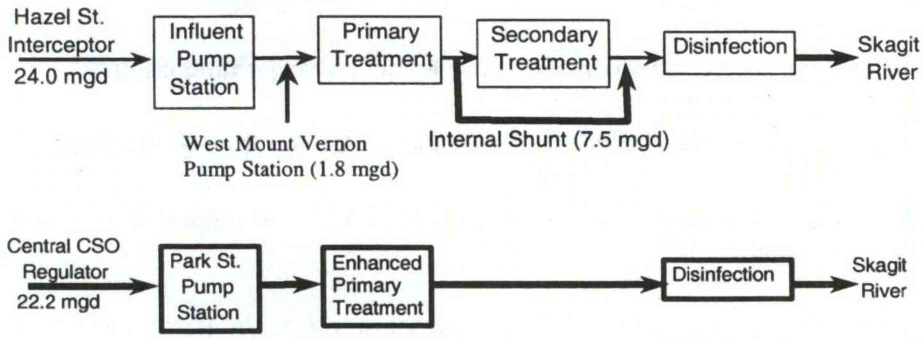
Treatment Recommendation

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. 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. 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. Construct a UV disinfection system Construct a 750 LF of 36-inch sewer. Upgrade Park Street Pump Station Construct 1500 LF of 30-inch force main Construct a CSO effluent pump station 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.

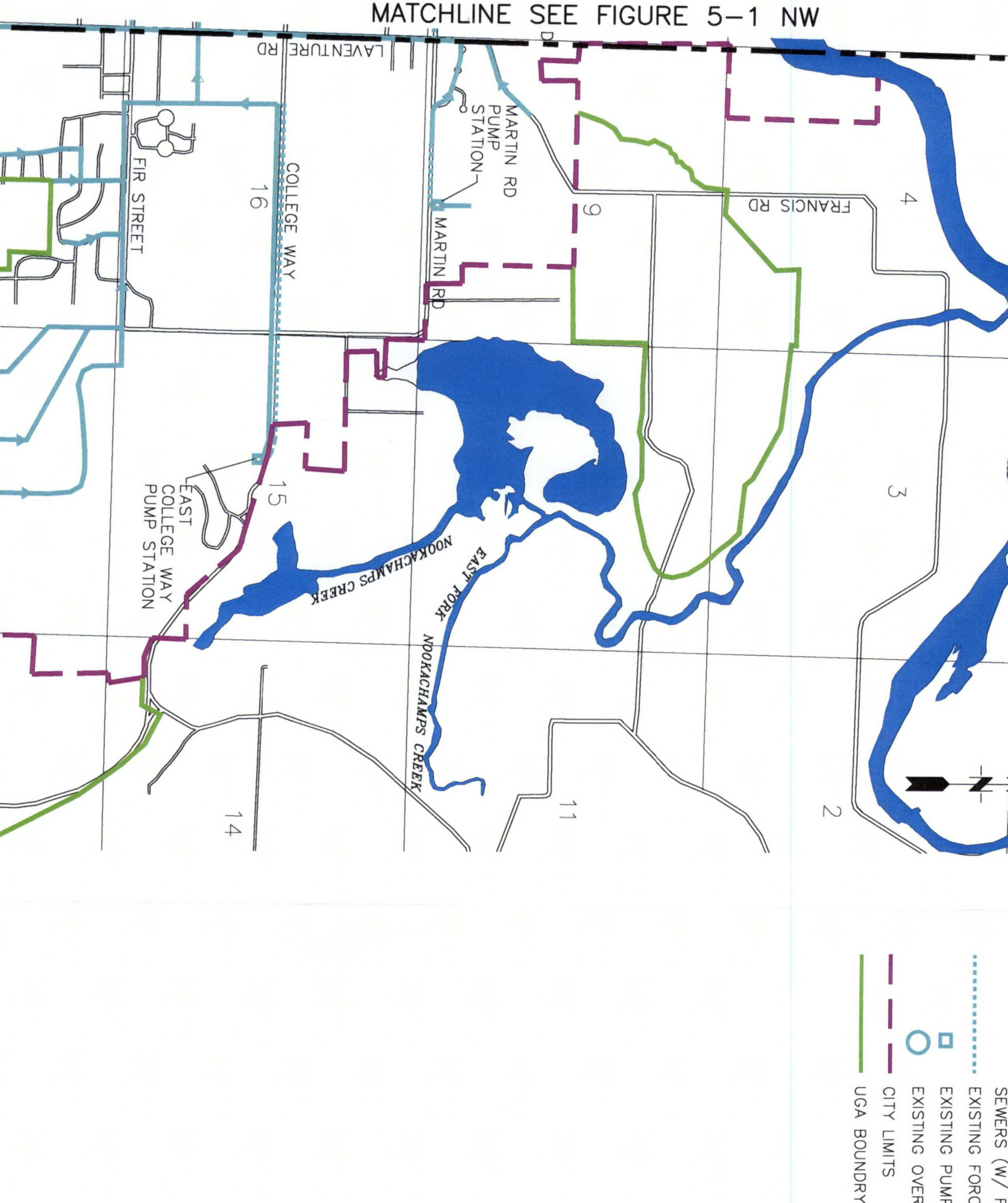
Portions of the downtown and older areas are served by combined sewers. Separate storm sewers are provided in the newer developed areas. The wastewater collection system was reviewed in 1994 and deficiencies in the system identified. Each year the City has allocated monies to repair known deficiencies.

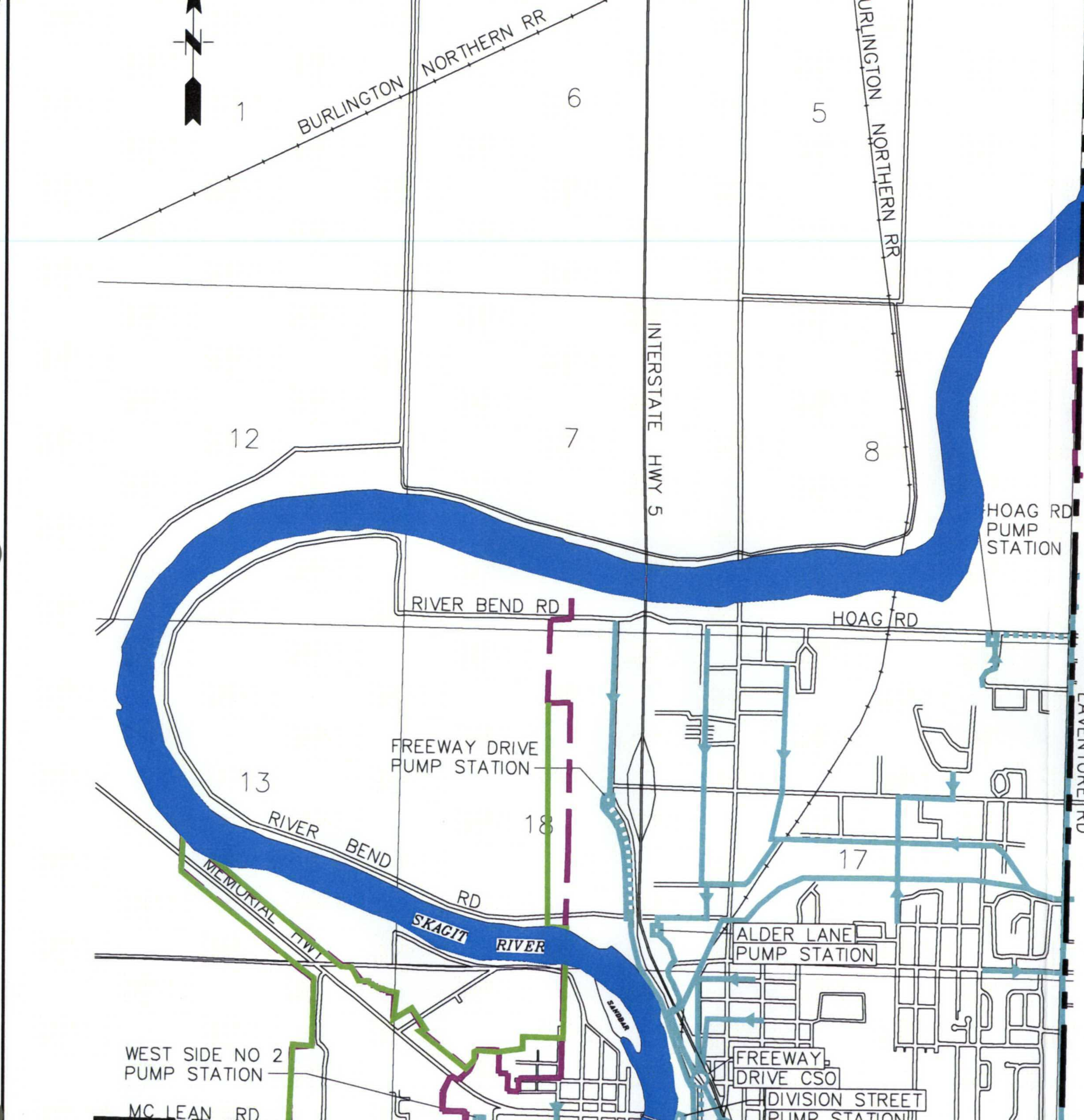
The wastewater collection system presently includes thirteen pump stations owned and operated by the City, Table 2-1. The City also maintains and operates three combined sewer overflow structures (Freeway Drive, Division Street, and Park Street) and two CSO/storm water pump stations (Division Street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

DATE: 02/12/03

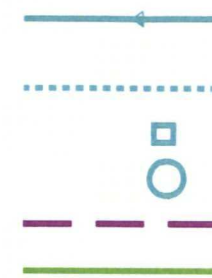
1
36
25
24
6
31
30
5
32
MAPLE LANE PUMP STATION
PARK STREET PUMP STATION
SEWAGE TREATMENT PLANT
DIKE RD
PARK STREET CSO
RIVER
HAZEL STREET PUMP STATION
BRITT RD
BURLINGTON NORTHERN RR
DIVISION STREET CSO
19TH STREET PUMP STATION
MATCHLINE SEE FIGURE 5-1 SE
LEGEND:
EXISTING SEWERS
EXISTING
EXISTING
EXISTING

DATE: 02/12/03


MATCHLINE SEE FIGURE 5-1 NW
LAVENTURE RD
MARTIN RD PUMP STATION
FRANCIS RD
COLLEGE WAY
FIR STREET
MARTIN RD
16
9
4
3
15
2
11
14
EAST COLLEGE WAY PUMP STATION
NOOKACHAMPS CREEK
EAST FORK NOOKACHAMPS CREEK
SEWERS (W/ F
EXISTING FORC
EXISTING PUMP
EXISTING OVER
CITY LIMITS
UGA BOUNDRY



MATCHLINE SEE FIGURE 5-1 NE



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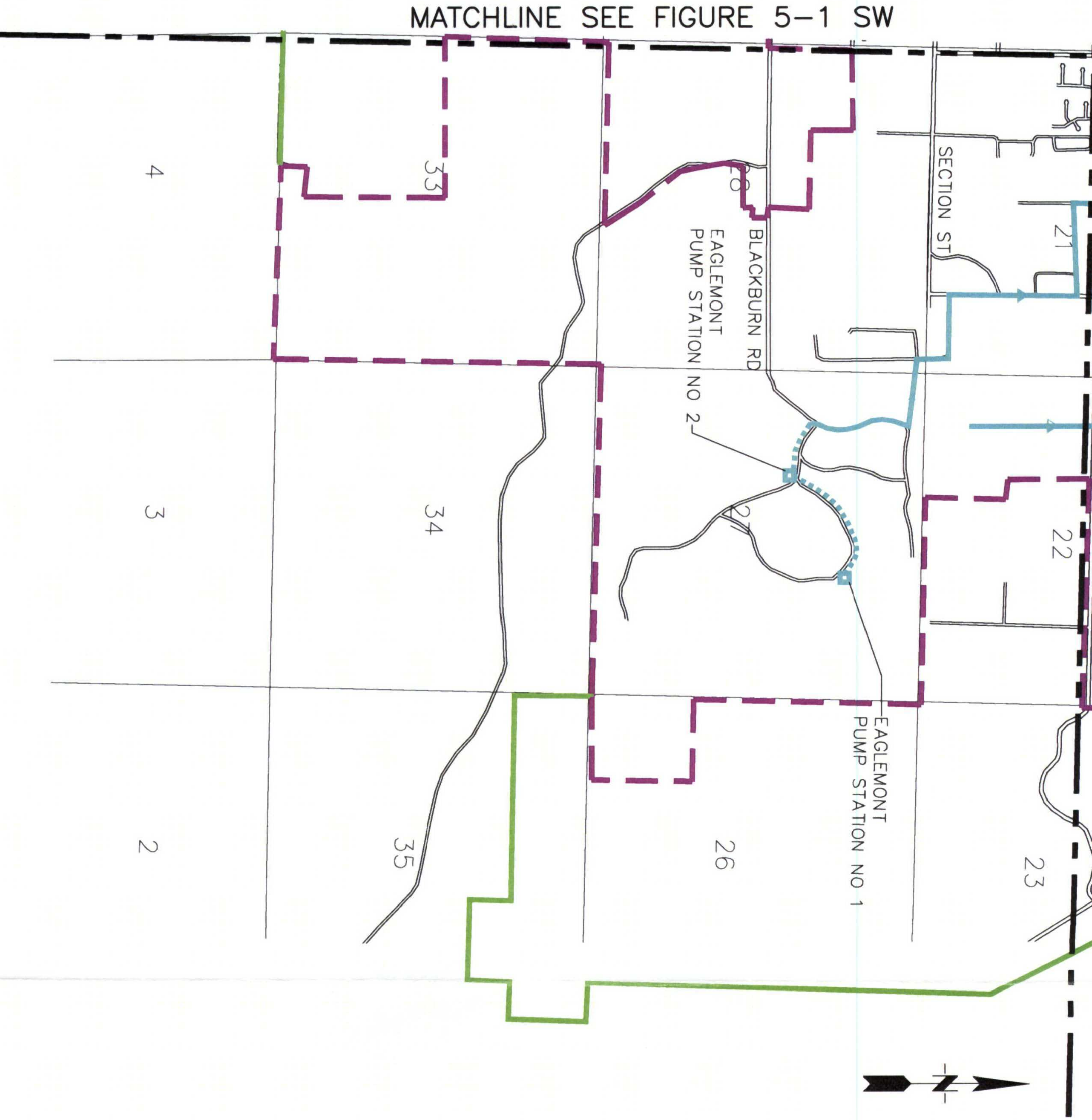
MATCHLINE SEE FIGURE 5-1 SW
4
33
28
21
SECTION ST
BLACKBURN RD
EAGLEMONT
PUMP STATION NO 2
3
34
27
22
EAGLEMONT
PUMP STATION NO 1
2
35
26
23

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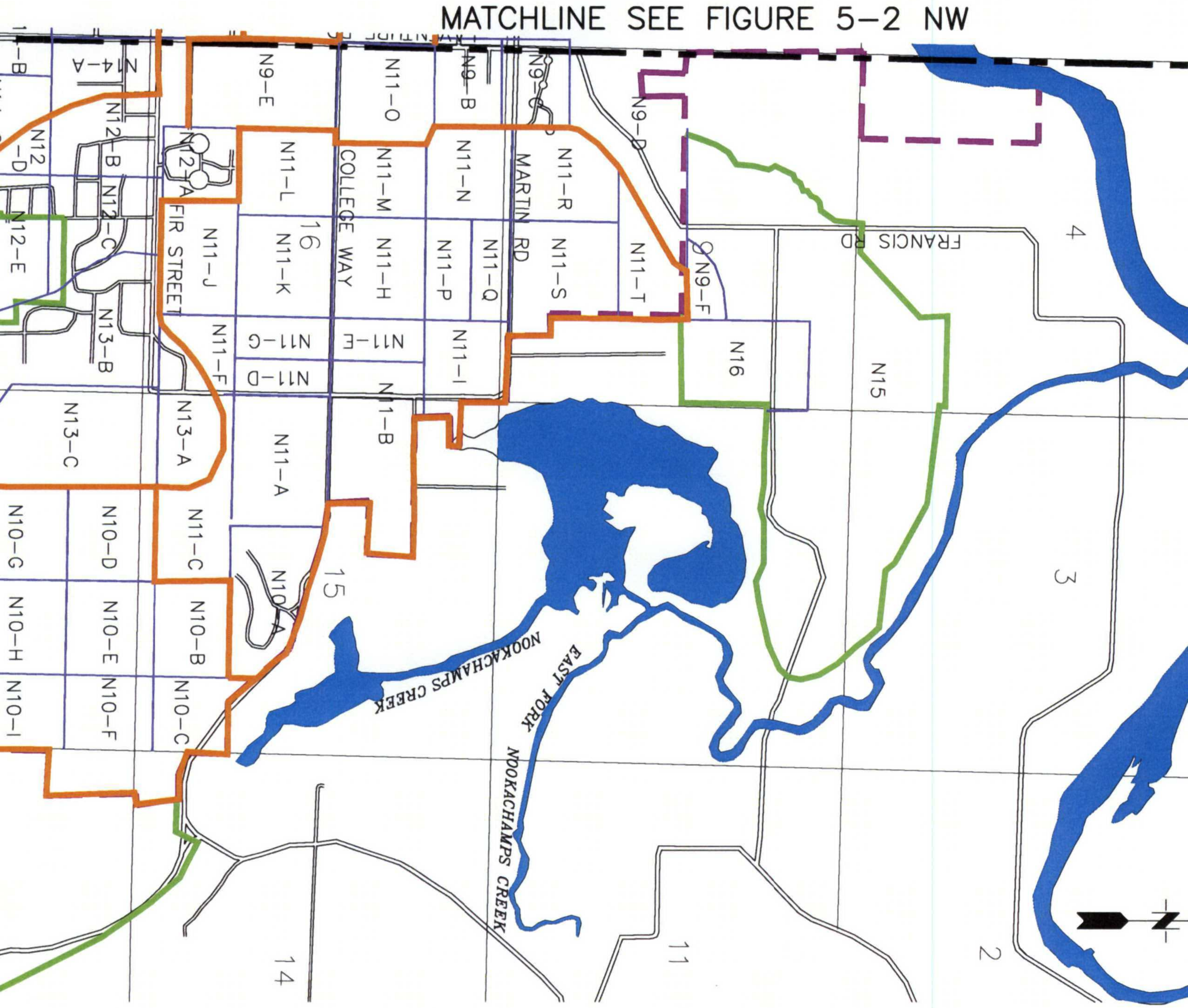

MATCHLINE SEE FIGURE 5-2 NW
FRANCIS RD
MARTIN RD
COLLEGE WAY
FIR STREET
NOOKACHAMPS CREEK
EAST FORK NOOKACHAMPS CREEK
N9-E
N9-B
N9-D
N9-F
N11-O
N11-N
N11-R
N11-L
N11-M
N11-S
N11-T
N11-K
N11-J
N11-H
N11-P
N11-Q
N11-G
N11-E
N11-I
N11-F
N11-D
N11-B
N11-A
N11-C
N12-B
N12-C
N12-D
N12-E
N13-B
N13-A
N13-C
N14-A
N15
N16
N10-A
N10-B
N10-C
N10-D
N10-E
N10-F
N10-G
N10-H
N10-I
2
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4
11
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16

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MC LEAN RD
MEMORIAL HY
RIVER BEND RD
RIVER BEND RD
SKAGIT RIVER
SANDBAR
INTERSTATE 5
BURLINGTON NORTHERN
BURLINGTON NORTHERN
HOAG RD
LAVENTURE RD
W2
N3
N-2
N4-A
N4-B
N4-C
N4-D
N5-A
N5-B
N5-C
N5-D
N5-E
N6
N7-A
N7-B
N7-C
N7-D
N7-E
N7-F
N8
N9-A
N1-A
N1-B
1
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18
MATCHLINE SEE FIGURE 5-2 NE
LEGE

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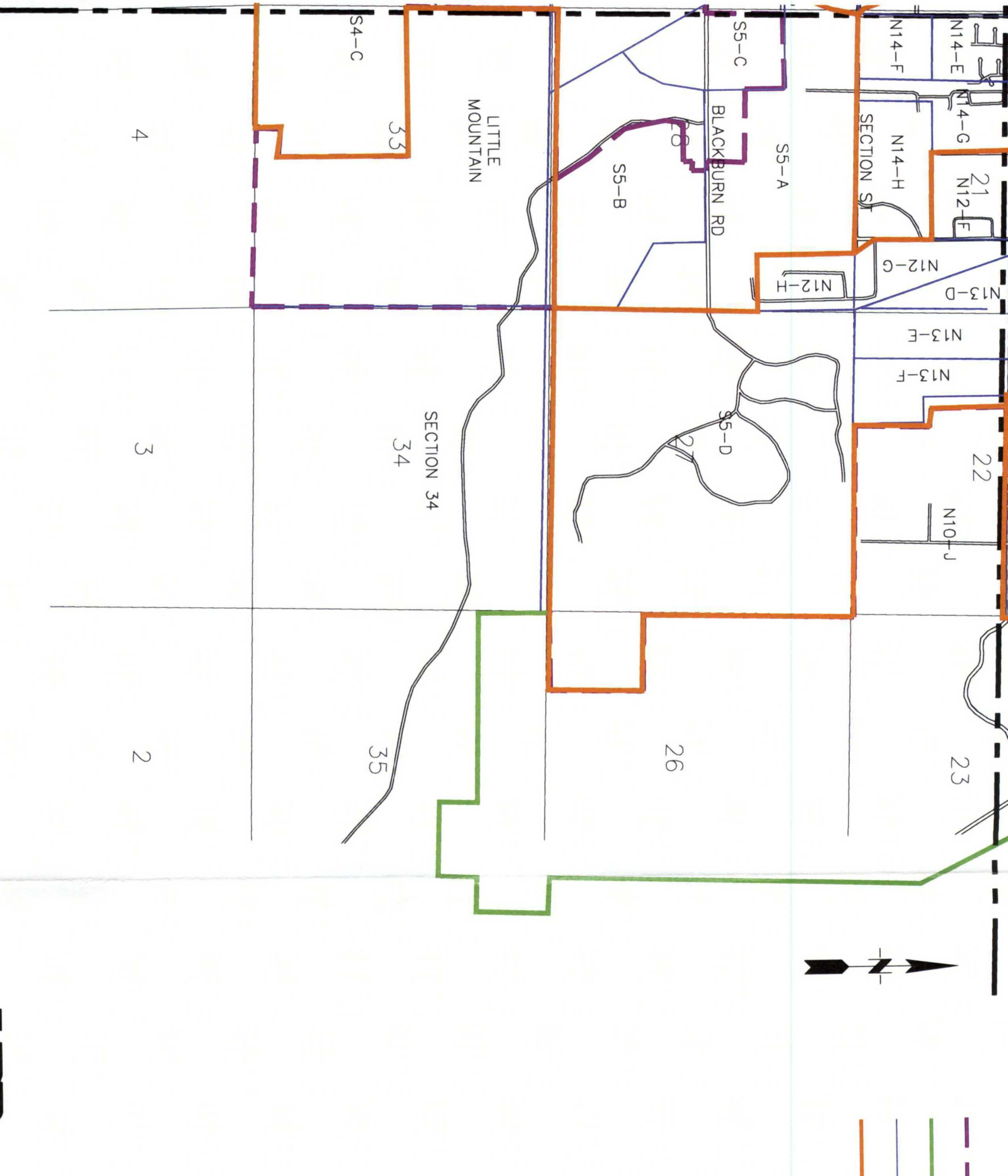
MATCHLINE SEE FIGURE 5-2 SW



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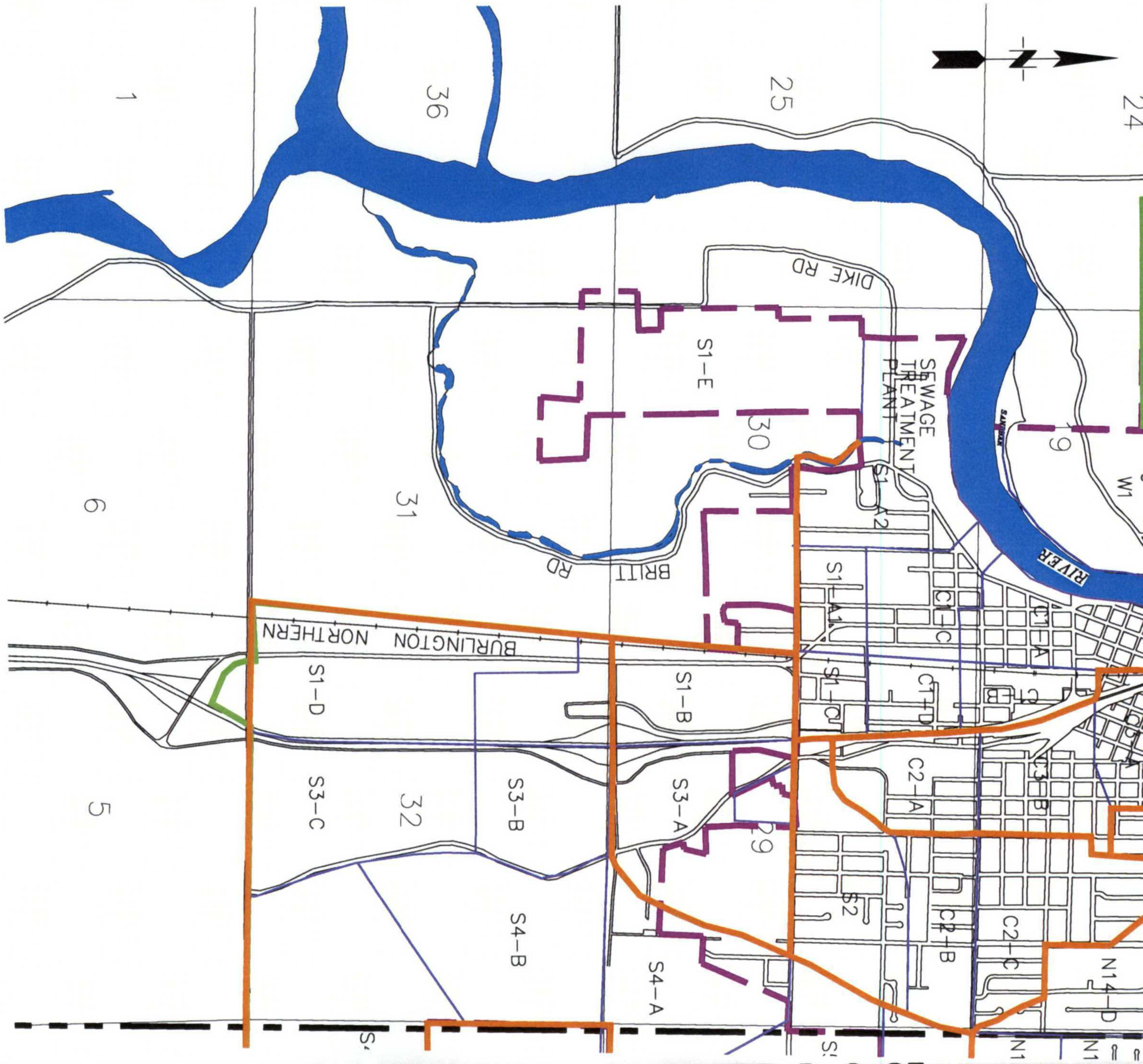

MATCHLINE SEE FIGURE 5-2 SE

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.

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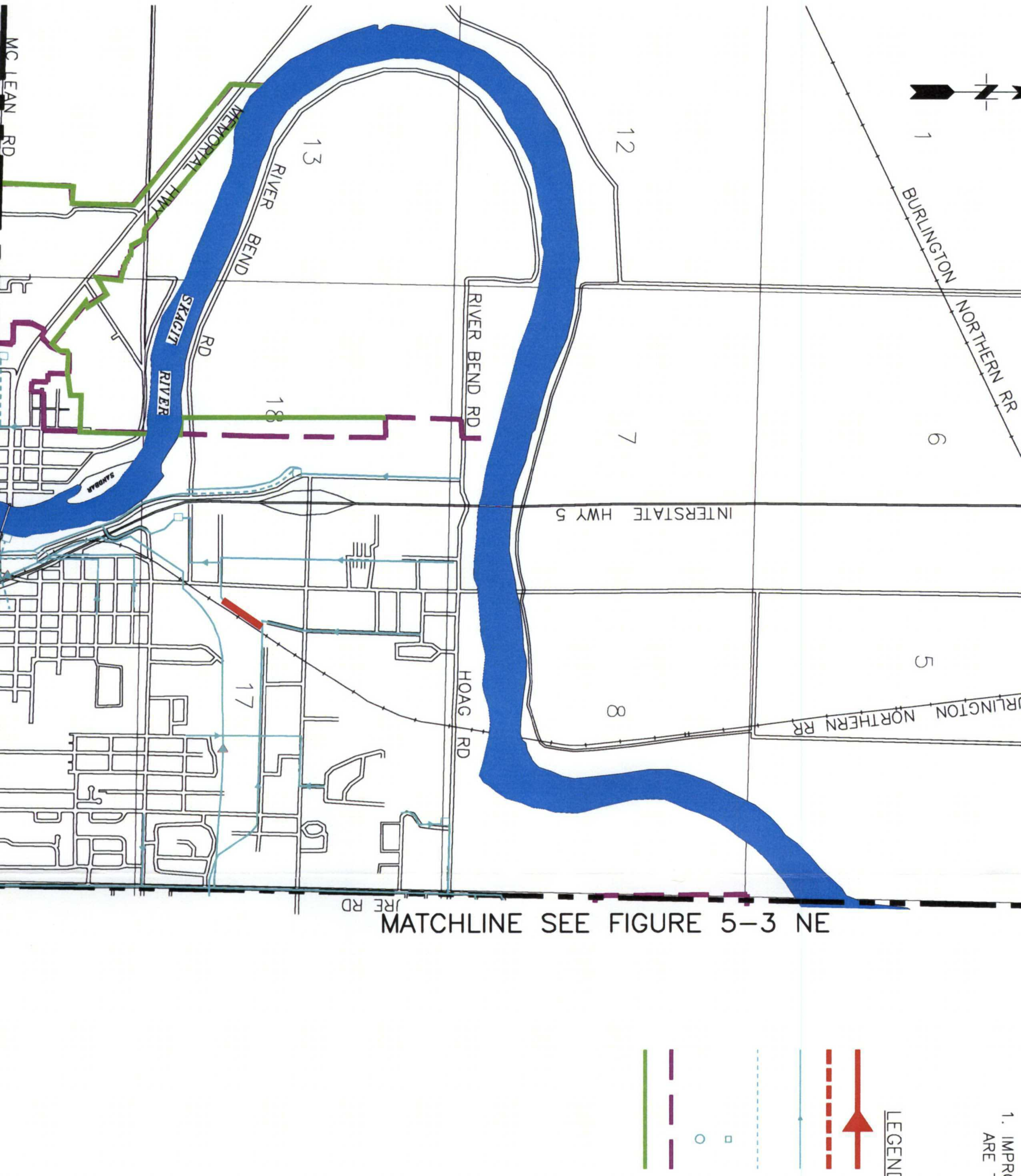

MC LEAN RD
MEMORIAL HWY
RIVER BEND RD
SKAGIT RIVER
SANDBAR
RIVER BEND RD
HOAG RD
INTERSTATE HWY 5
BURLINGTON NORTHERN RR
URLINGTON NORTHERN RR
1
5
6
7
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12
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17
18
JRE RD
MATCHLINE SEE FIGURE 5-3 NE
LEGEND
1. IMPRO
ARE

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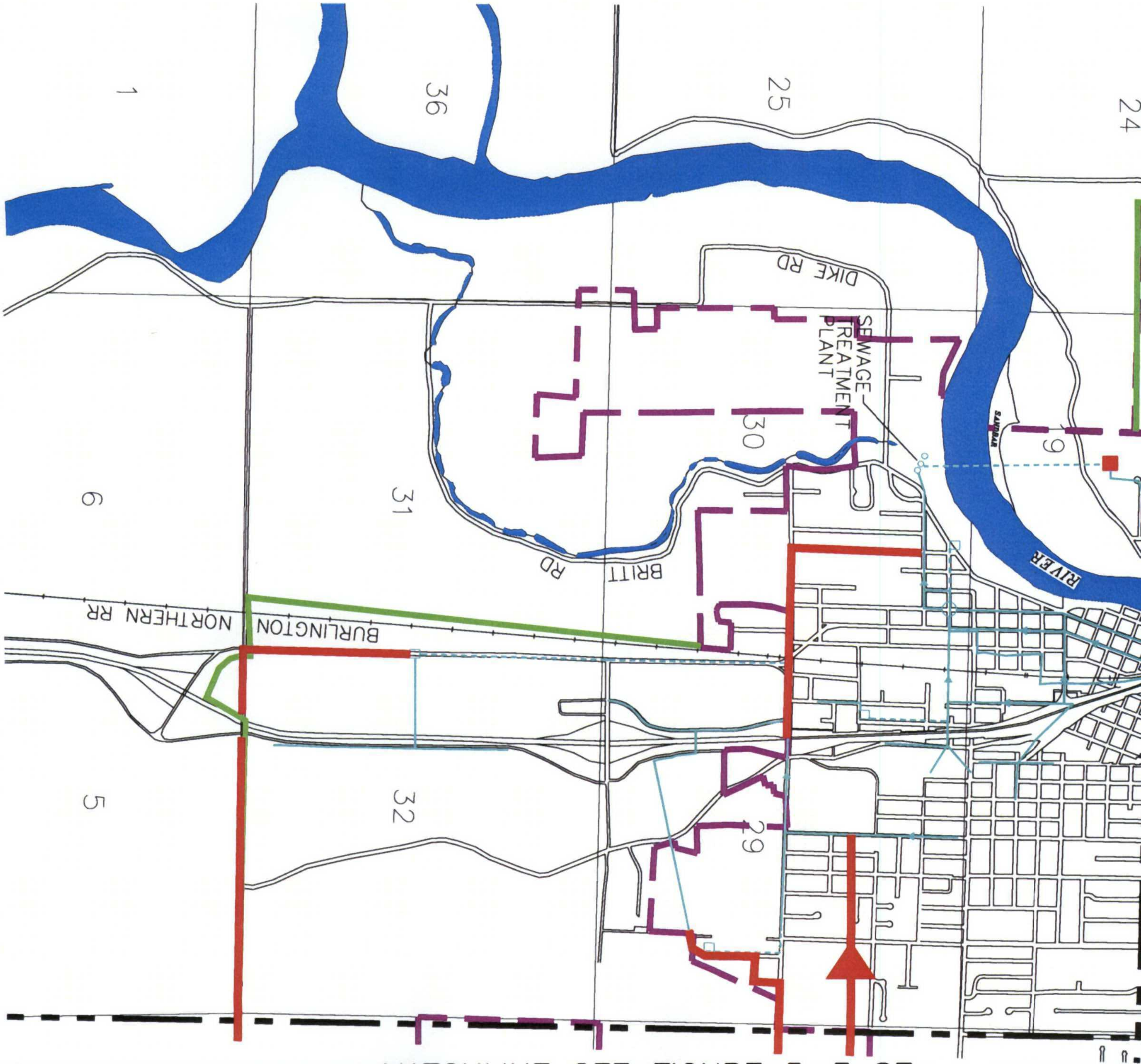
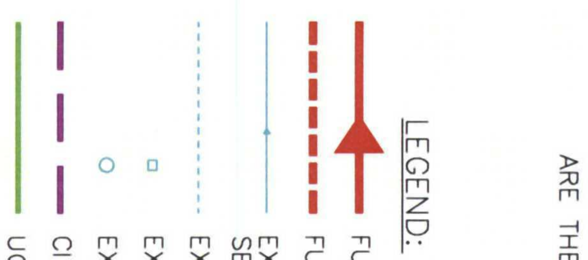


MATCHLINE SEE FIGURE 5–3 SE

ARE THE



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MATCHLINE SEE FIGURE 5-3 SW

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.

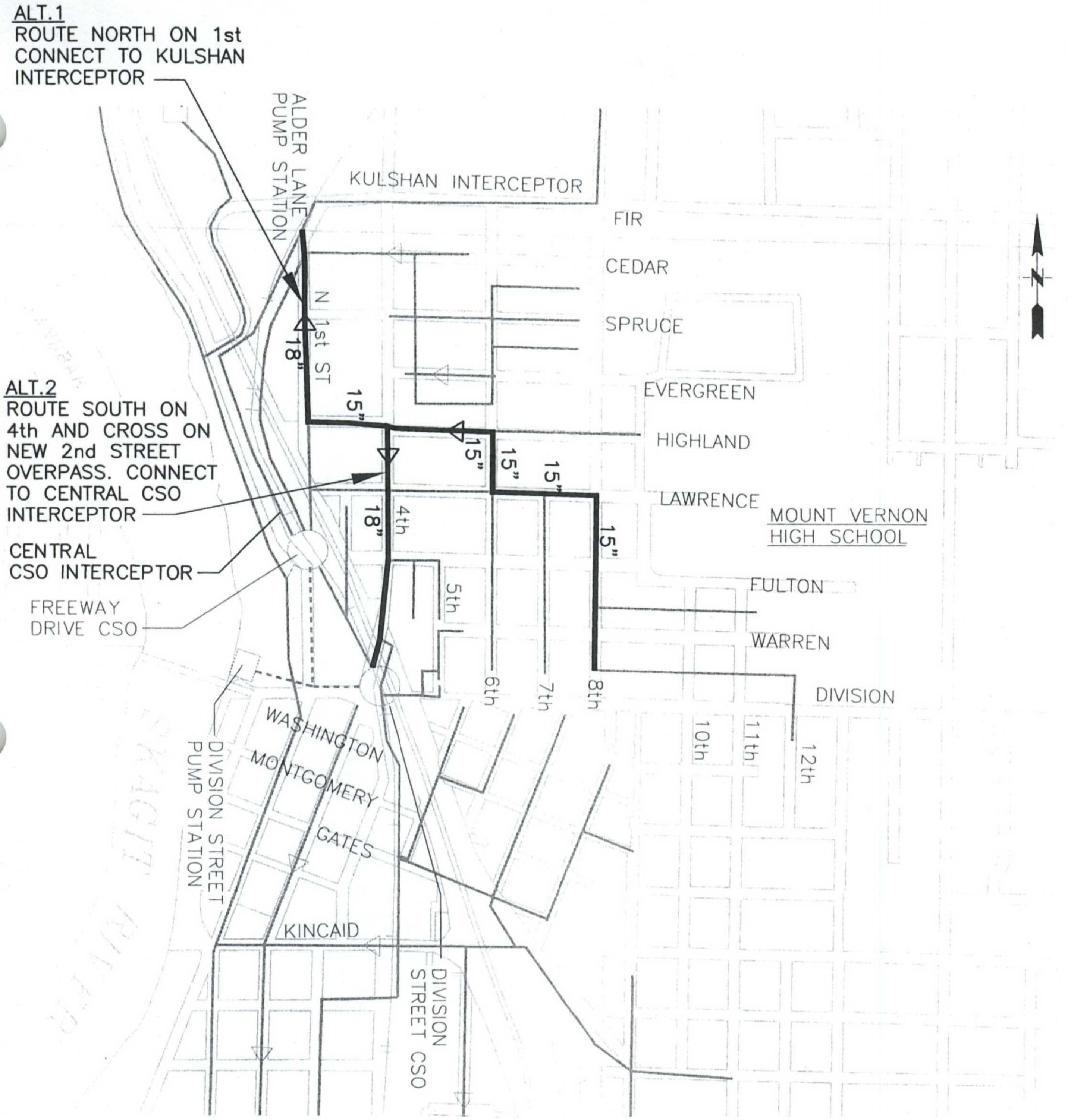
Interstate 5 Crossings		
No.	Location	Condition and Recommended Improvement
		necessary.
8.	Section Street at Wells Nursery	Condition unknown. This 16-inch provides service to only one connection, Wells Nursery. There is also a documented steady flow
9.	Park Street at South Side of Wells Nursery	Condition 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.



LEGEND:

MAIN COLLECTION SEWERS (W/ FLOW ARROW)

PUMP STATION

OVERFLOW SEWER

OVERFLOW STRUCTURES

HDR
Engineering, Inc.

Project Title
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

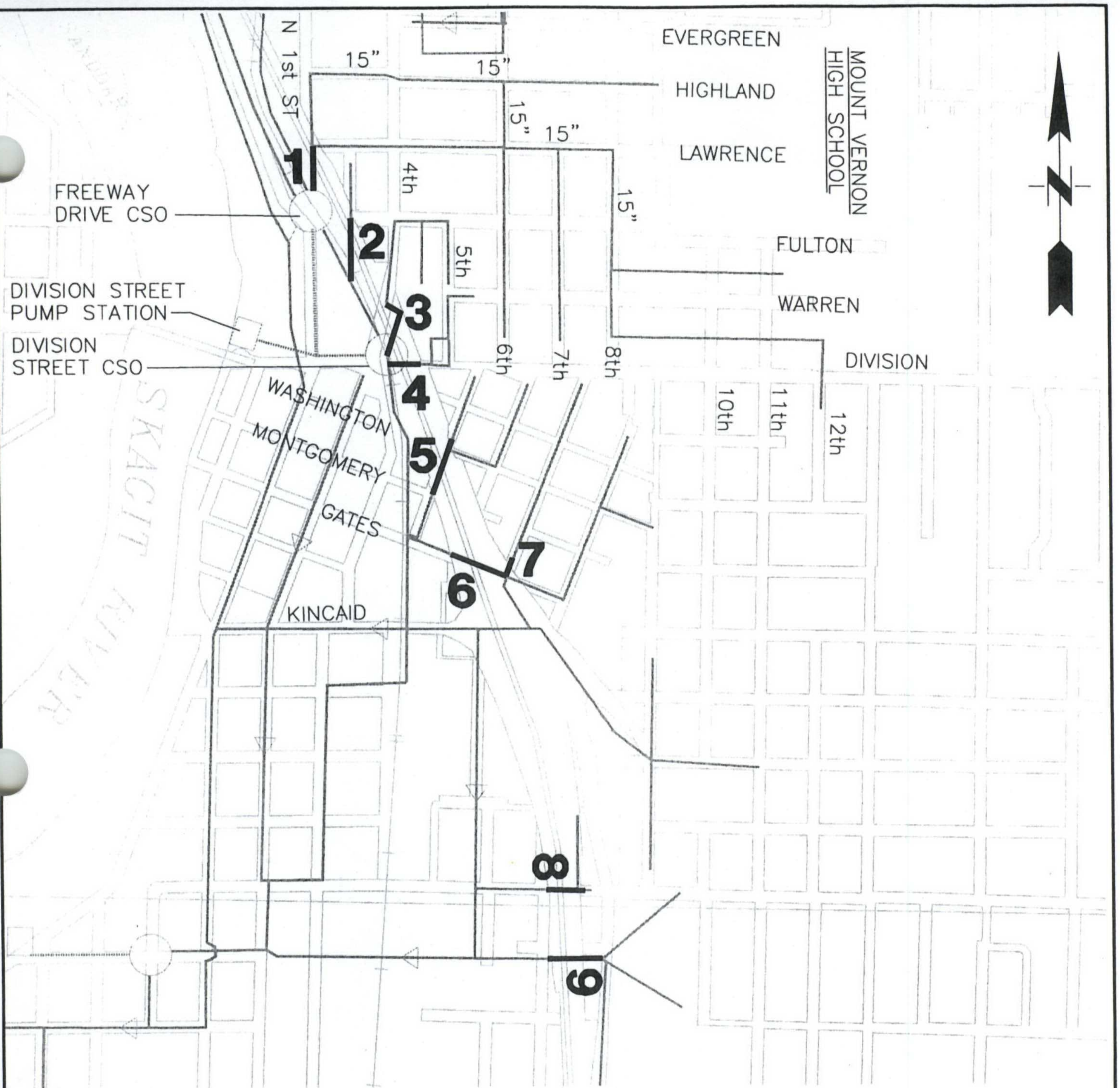
Date
FEBRUARY 2003

Sheet Title
NORTH 8th STREET IMPROVEMENTS

Figure No.
5-4

DATE: 02/12/02

FILENAME: FIG5-



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

HDR Engineering, Inc.

DATE: 02/12/03

FILENAME: FIG5-5.D

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 NO. 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 utilities is to avoid the use of gaseous chlorine when planning new facilities. Presently the City utilizes 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:

Parameter	Maximum consecutive 3-Day Average	Daily Maximum
	(rolling average)	
Flow	N/A	0.63 mgd
BOD_5	1300 lb/day	N/A
TSS	750 lb/day	N/a
FOG	N/A	100 mg/L
PH	5.0 to 11.0	

At the maximum flow of 0.63 mgd, 1300 lb/day of BOD_5 equates to a concentration near 250 mg/L, while 750 lb/day of TSS yields a concentration of 143 mg/L. However production day flows are often around 0.55 mgd, which results in allowable BOD_5 and TSS concentrations around 285 mg/L and 165 mg/L, respectively.

WASTEWATER DATA

Average monthly flow, BOD and suspended solids for the wastewater from DVF for the 14 months from May 1999 through July 2000 are presented in Table 6-1.

Table 6-1

Historical Flows and Loads for Draper Valley Farms, Inc.			
Month	Average Monthly Flow (mgd)	Average Monthly BOD (lbs/day)	Average Monthly SS (lbs/day)
May-99	0.491	953	523
Jun-99	0.505	1065	552
Jul-99	0.495	1179	658
Aug-99	0.502	1088	598
Sep-99	0.452	1022	541
Oct-99	0.484	1208	598
Nov-99	0.408	940	396
Dec-99	0.334	783	381
Jan-00	0.5	901	470
Feb-00	0.424	889	505
Mar-00	0.425	699	528

Historical Flows and Loads for Draper Valley Farms, Inc.			
Month	Average Monthly Flow (mgd)	Average Monthly BOD (lbs/day)	Average Monthly SS (lbs/day)
Apr-00	0.454	480	403
May-00	0.429	872	547
Jun-00	0.39	484	354
Jul-00	0.408	678	462

In November 1999, Draper Valley Farms switched pretreatment chemical supply companies. The new company, CESCO Chemicals, Inc., made the temporary revisions necessary to feed ferric chloride and acid to improve protein recovery. After an initial startup period, much better results have been achieved. The most significant reductions have been in the average daily BOD discharges to the system. As shown Figure 6-1, monthly average discharges exceeded 1200 lbs. per day in October 1999 and have averaged less than 700 pounds per day for the period from December 1999 through July 2000. Average monthly suspended solids loadings are shown on Figure 6-2.

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

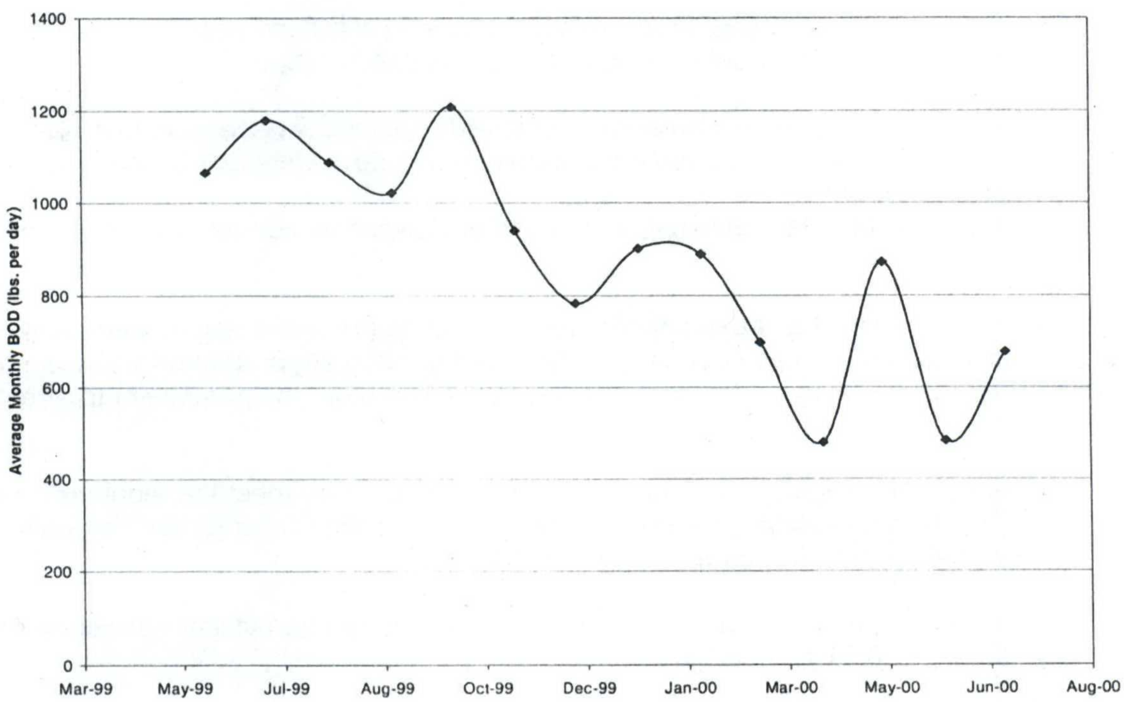
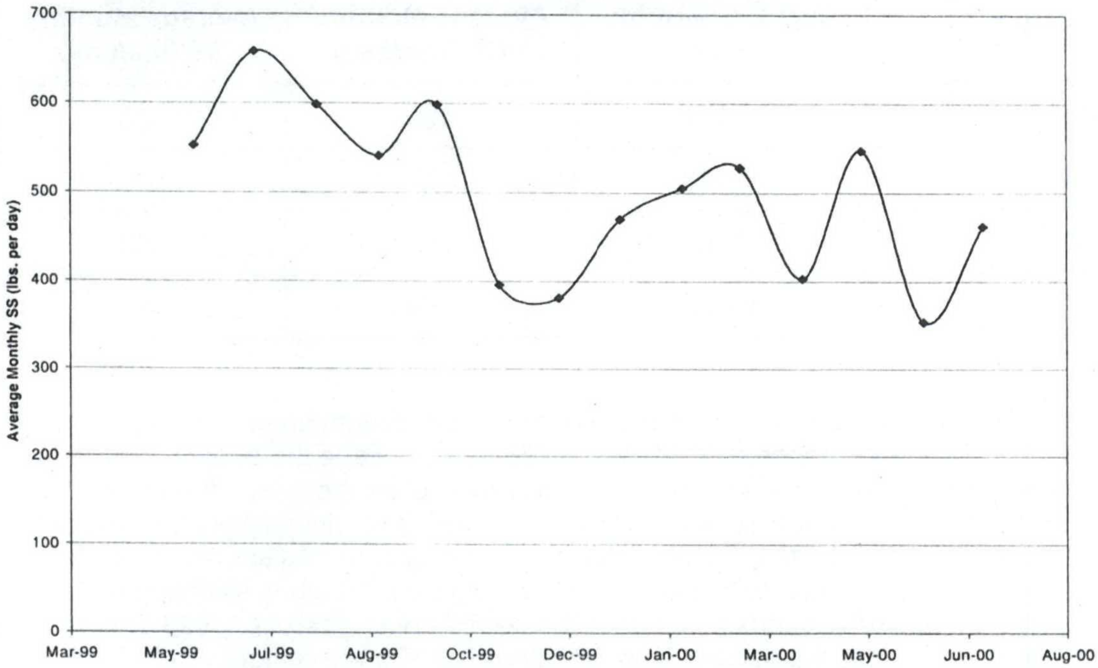


Figure 6-2 DVF Wastewater Discharges -Suspended Solids (lbs. per day)



AKART

Chapter 173-216 of the Washington Administrative Code (WAC) defines the State Waste Discharge Permit Program. Section 73-216-110 of the WAC states:

> "Any permit issued by the department shall specify conditions necessary to prevent and control waste discharges into the waters of the state, including the following, whenever applicable:
> All known, available, and reasonable methods of prevention, control, and treatment;"

The acronym of AKART has been adopted for Subsection (a) of this requirement. AKART is not specifically defined for the case of a poultry plant discharging pretreated wastewater to a city's sewerage system for treatment and disposal. Therefore the purpose of this section is to:

1. Define what is required for Draper Valley Farms (DVF) to meet the requirements of AKART as they discharge pretreated wastewater to the City of Mount Vernon's sewerage system for treatment and discharge.

2. Determine whether the existing pretreatment discharge permit limits are appropriate for these AKART technologies.

Although AKART is not specifically defined for the case of a poultry plant discharging pretreated wastewater to a city's sewerage system for treatment and disposal, in 1975 the EPA published a document entitled Development Document for Proposed Effluent Limitations Guidelines and New Source Performance Standards for the Poultry Segment of the Meat Product and Rendering Process Point Source Category. That document identified "Best Practicable Control Technology Currently Available" (BPT), "New Source Control Technology" (New Source), and "Best Available Technology Economically Achievable" (BAT). That document was written for poultry plants with direct wastewater discharges, but portions of those technology lists are still relevant to the definition of AKART for this particular case, even though 25 years old. Relevant portions of the EPA recommendations were compared with the DVF practices:

EPA Recommendations	**Draper Valley Farms Practices**
• Appoint a person with specific responsibility for water management. This person should have reasonable powers to enforce improvements in water and waste management.	• Yes
• Make all employees aware of good water management practices and encourage them to apply these practices.	• Yes
• Install and monitor flow meters in all major water use areas.	• Yes
• Determine or estimate water use and waste load strength from principal sources.	• Yes
• Control and minimize flow of freshwater at major outlets by installing properly-sized spray nozzles and by regulating pressure on supply lines.	• Yes
• Install "demand" valves on all freshwater outlets.	• Yes, with the exception of an open hose without a nozzle near the inside/outside spray cabinet.
• Install press-to-operate valves on hand washers.	• No, but minimal number of hand washers
• Use high-pressure, low-volume spray nozzles or steam-augmented systems for plant washdown.	• Yes. Separate high-pressure (approx. 500 psi) sanitation system
• Shut off all unnecessary water flow during work breaks.	• Generally yes, except as follows 1. Water to foot plucking machine ran during noon break; other plucking machines ran less.

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 1401 lbs/day	45 mg/L 2102 lbs/day
Total Suspended Solids (TSS)	30 mg/L 1401 lbs/day	45 mg/L 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.

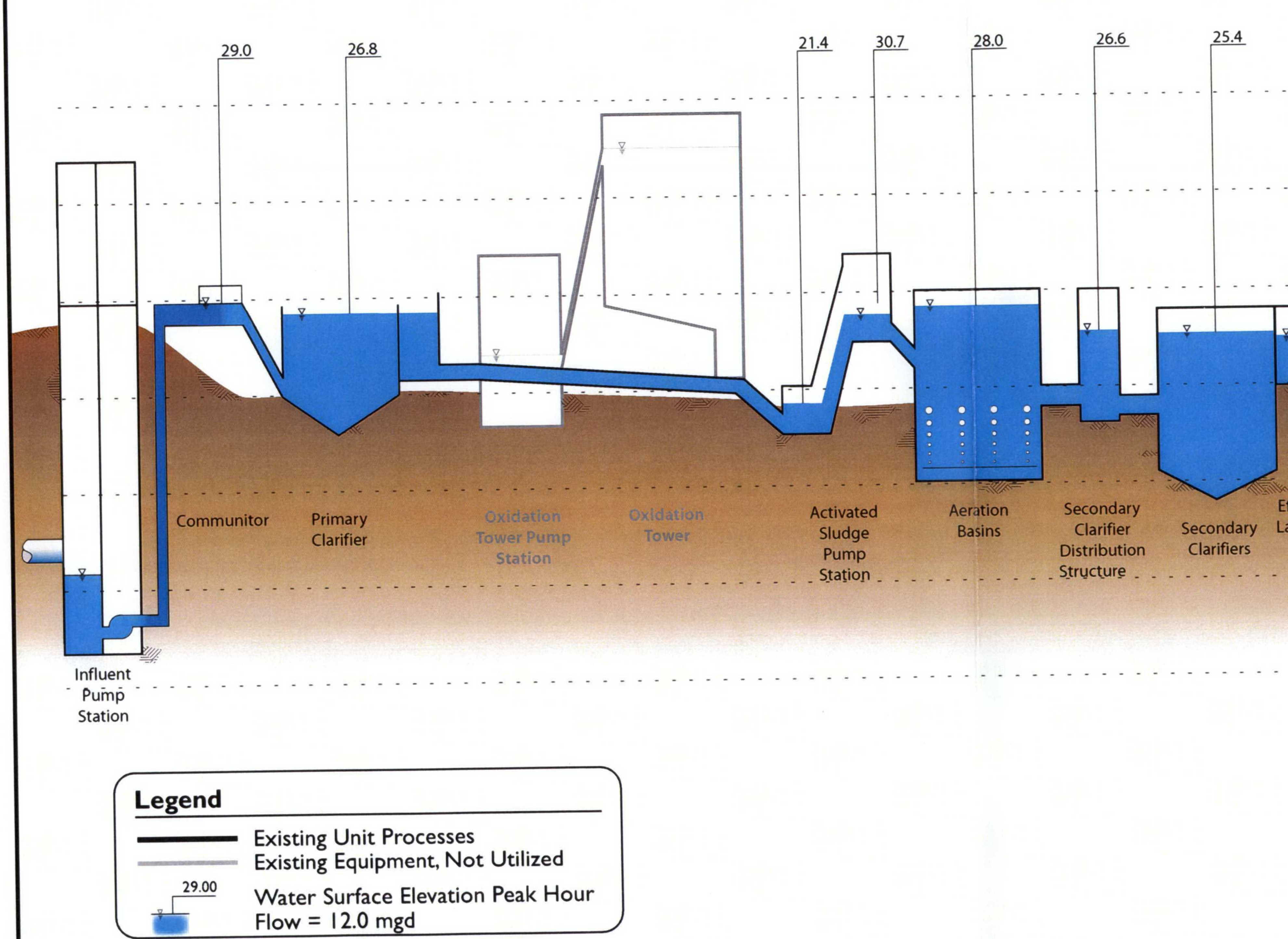
Table 7-3

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

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


29.0
26.8
21.4
30.7
28.0
26.6
25.4
Influent Pump Station
Communitor
Primary Clarifier
Oxidation Tower Pump Station
Oxidation Tower
Activated Sludge Pump Station
Aeration Basins
Secondary Clarifier Distribution Structure
Secondary Clarifiers
Legend
Existing Unit Processes
Existing Equipment, Not Utilized
29.00
Water Surface Elevation Peak Hour Flow = 12.0 mgd

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 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 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 (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.

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 operating 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.

Process	Existing Facility Description	Size or Capacity	Criteria Flow	Parameter	Capacity Criteria	Reference	Value with BOD removal[1]	Value with Nitrification[1]
Primary Clarifier	1-primary clarifier	80 ft diameter	MM	OFR	1,000 gpd/sf	DOE Standard	2,100 gpd/sf	Same as BOD
		10 ft side water depth	PH	OFR	2,500 gpd/sf	DOE Standard	3,800 gpd/sf	Same as BOD
		5,000 sf						
		0.4 MG	MM	HRT	2.5 hr	DOE Standard	0.9 hr	Same as BOD
Aeration Basins	3-plug flow aeration basins	61 ft length, 42 ft width, 17.5 ft SWD	MM	MLSS	2,500 mg/L	Stress testing	3,000 mg/L	6,400 mg/L
		0.33 MG each, 1.0 MG total	MD	MLSS	2,700 mg/L	Stress testing	2,800 mg/L	7,200 mg/L
	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
			MD	OUR	36 mg/L-hr[2]	HDR Standard	45 mg/L-hr	80 mg/L-hr
			PH	OUR	54 mg/L-hr[2]	HDR Standard	56 mg/L-hr	113 mg/L-hr
Aeration System—Blowers	4-centrifugal	4,100 scfm each	MD	SCFM	12,300	None	5,900 scfm	12,800 scfm
		12,300 scfm with 1 out of service	PH	SCFM	12,300	None	9,000 scfm	16,600 scfm
Secondary Clarifiers	2-secondary clarifiers	85-ft diameter	MD	HRT	<2 hr	HDR Standard	1.5 hr	1.5 hr
		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
		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
		1-0.64 MGD	MD	SLR	40 lb/d-sf[2]	DOE Standard	32 lb/d-sf	38 lb/d-sf
			PH	SLR	40 lb/d-sf[2]	DOE Standard	51 lb/d-sf	51 lb/d-sf
Gravity Thickener	1-thickener	22-ft diameter 10-ft SWD	MM	OFR	700 gpd/sf	DOE Standard	261 gpd/sf	353 gpd/sf
DAF Thickener	1- DAF Thickener	40-ft diameter 11 ft SWD 1,260 sf	MM	SLR	2.5 lb/hr-sf	DOE Standard	4.0	3.5
Anaerobic Digester	1-anaerobic digester	60 ft diameter,	MM	SRT	15 days	EPA Standard	28 d	33 d
		34 ft SWD,	MM	SLR	140 lbVSS/kcf-d	WEF MOP8	80 lbVSS/kcf-d	70 lbVSS/kcf-d
		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.

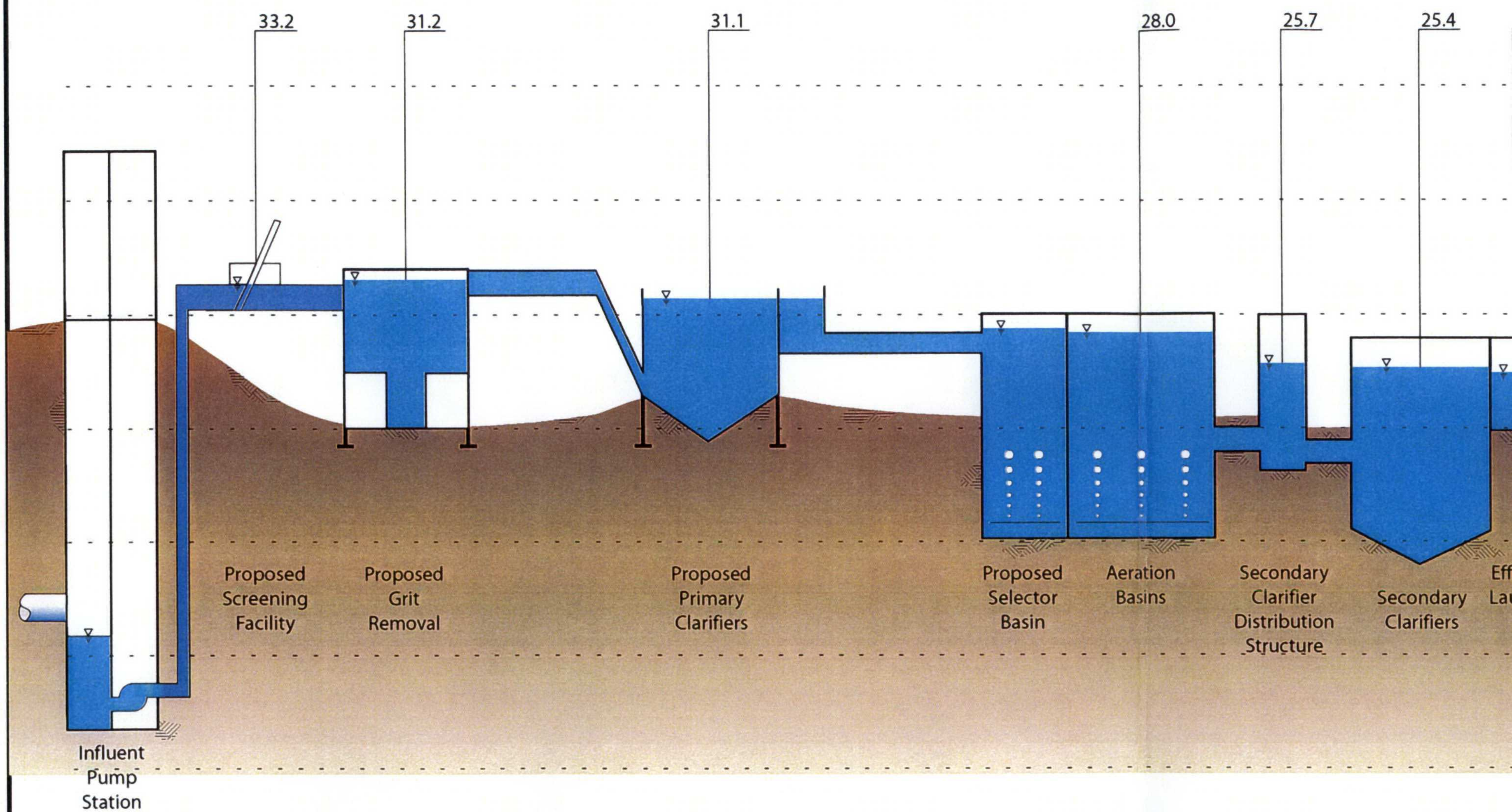
Alternative A - Existing WWTP Hydraulics

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

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

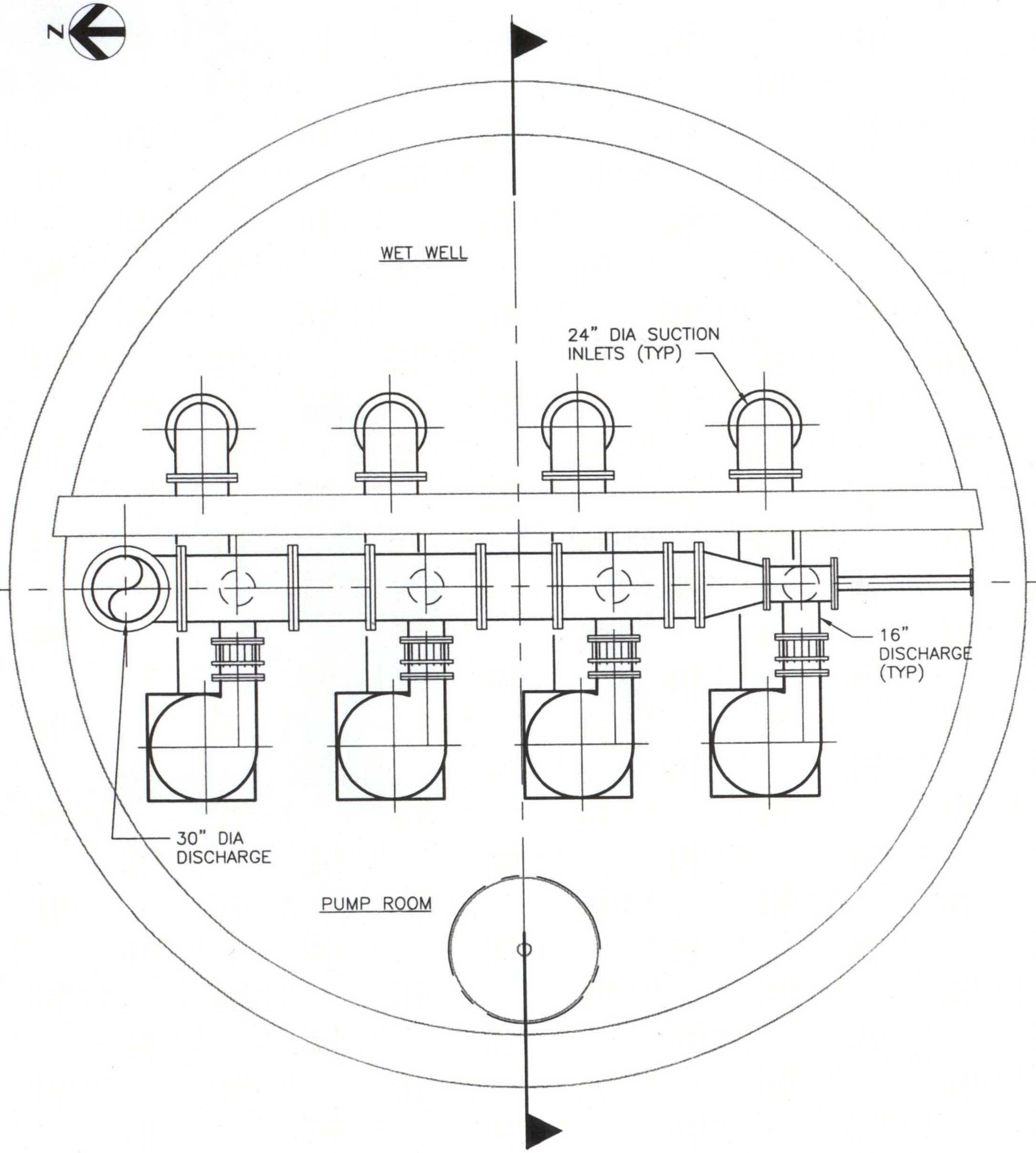
Influent Pump Station: Alternative A Cost Estimate (Upgrading Existing Wetwell/Drywell Pump Station)				
Item	**Quantity**	**Unit**	**Unit Cost**	**Cost**
Bypass Pumping	1	LS	$50,000	$50,000
Replace Existing Pumps	4	EA	$65,000	$260,000
Replace Existing Piping	1	LS	$80,000	$80,000
Forcemain	1	LS	$200,000	$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	**Quantit y**	**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



DATE: 02/11/03

FILENAME: FIG9-2

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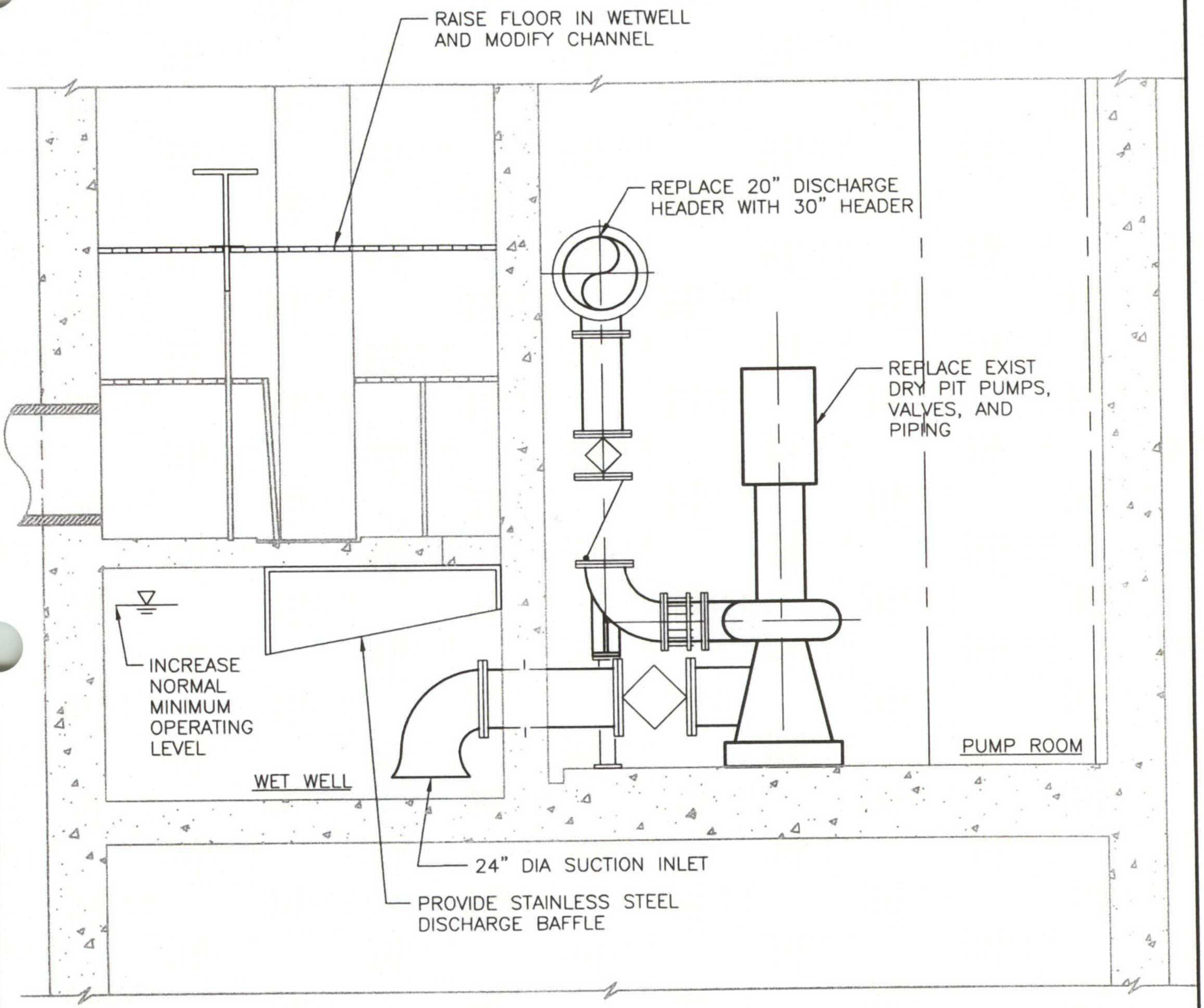
Project Title
MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date
FEBRUARY 2003

Sheet Title
INFLUENT PUMP STATION UPGRADE ALTERNATIVE A-PLAN

Figure No.
9-2



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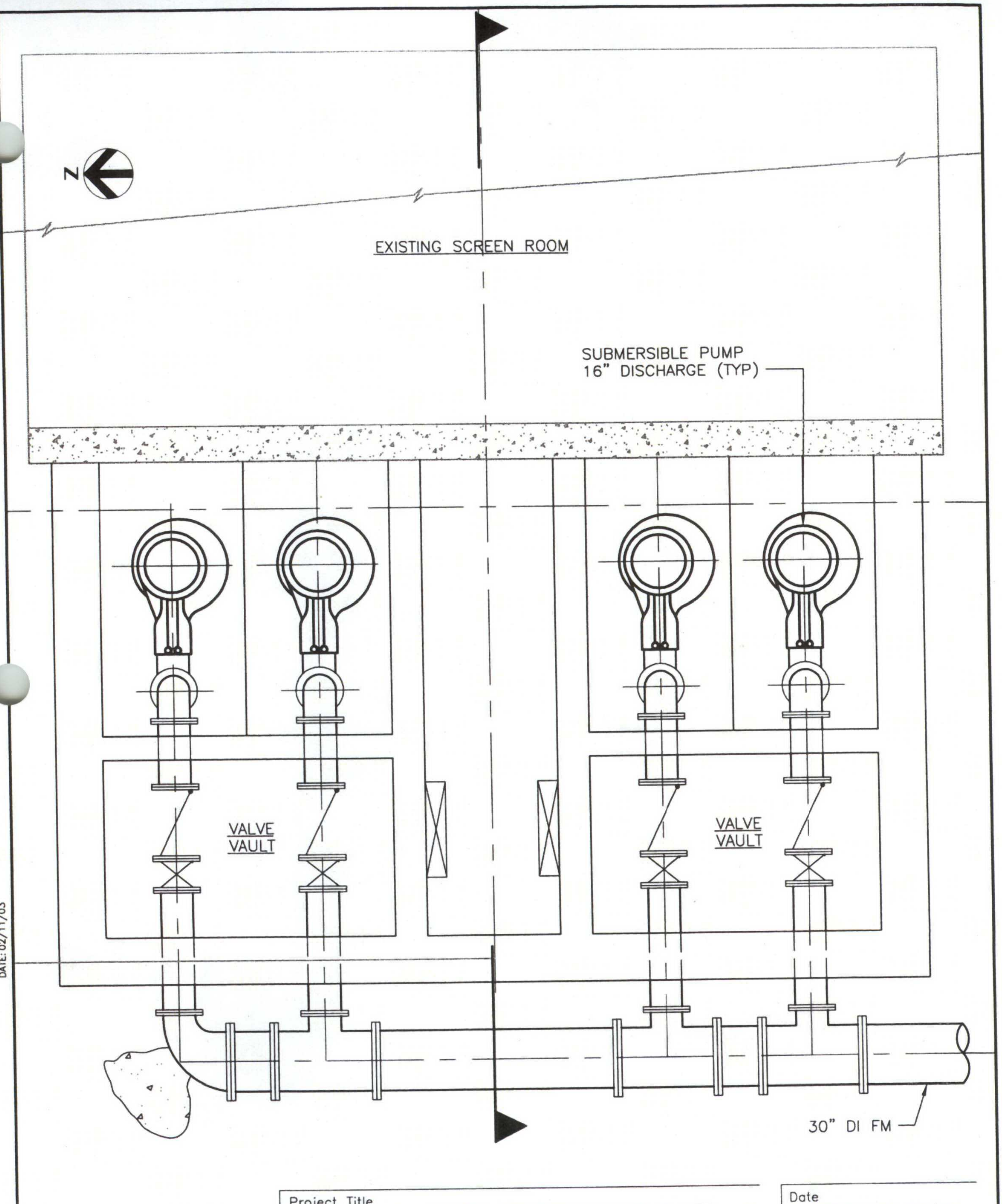
HDR Engineering, Inc.

Project Title: MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date: FEBRUARY 2003

Sheet Title: INFLUENT PUMP STATION UPGRADE ALTERNATIVE A-TYPICAL SECTION

Figure No. 9-3



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MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date
FEBRUARY 2003

Sheet Title
INFLUENT PUMP STATION UPGRADE ALTERNATIVE B–PLAN

Figure No.
9–4

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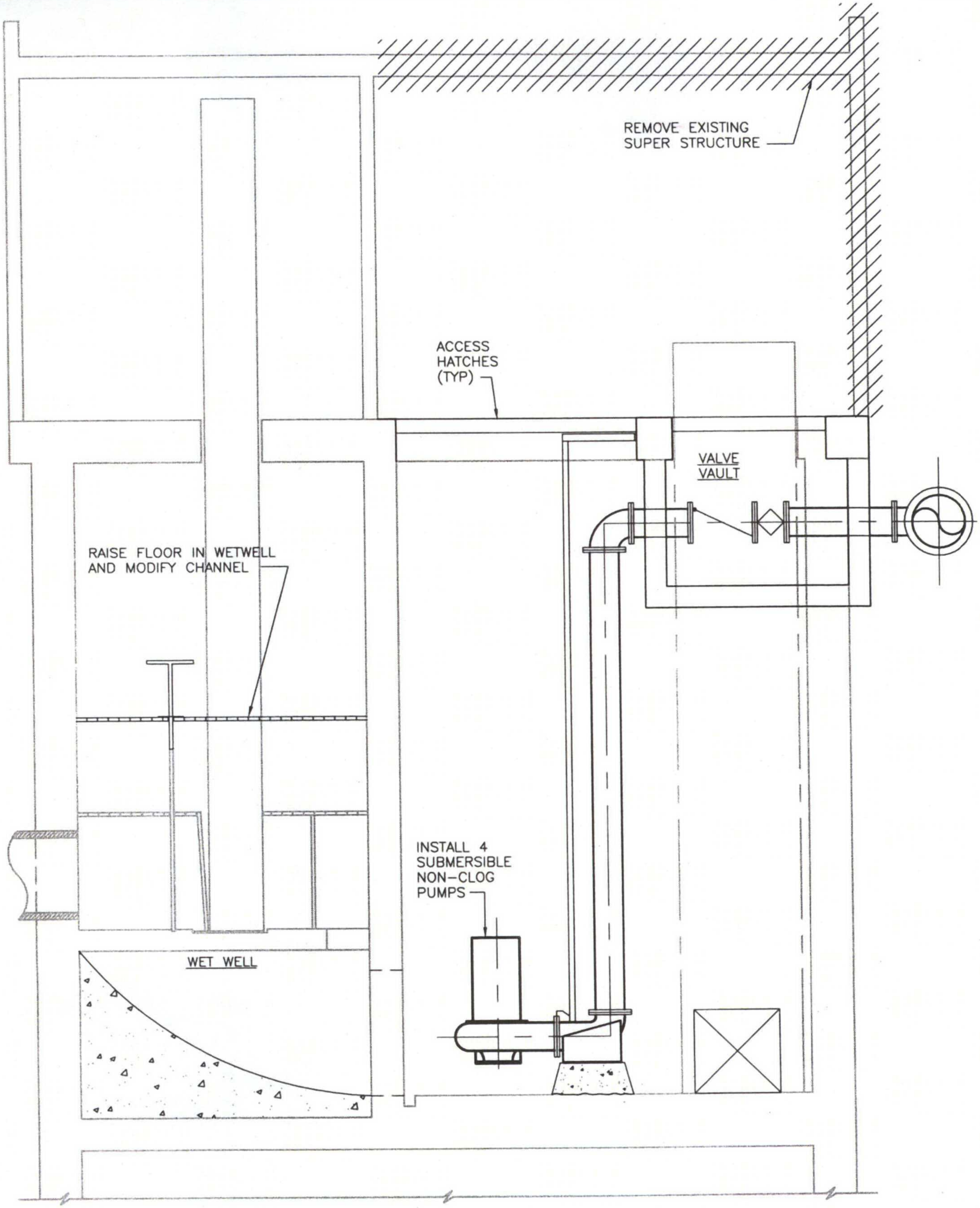

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MOUNT VERNON COMPREHENSIVE SEWER PLAN UPDATE

Date
FEBRUARY 2003

Sheet Title
INFLUENT PUMP STATION UPGRADE
ALTERNATIVE B–TYPICAL SECTION

Figure No.
9–5

DATE: 02/11/03

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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.

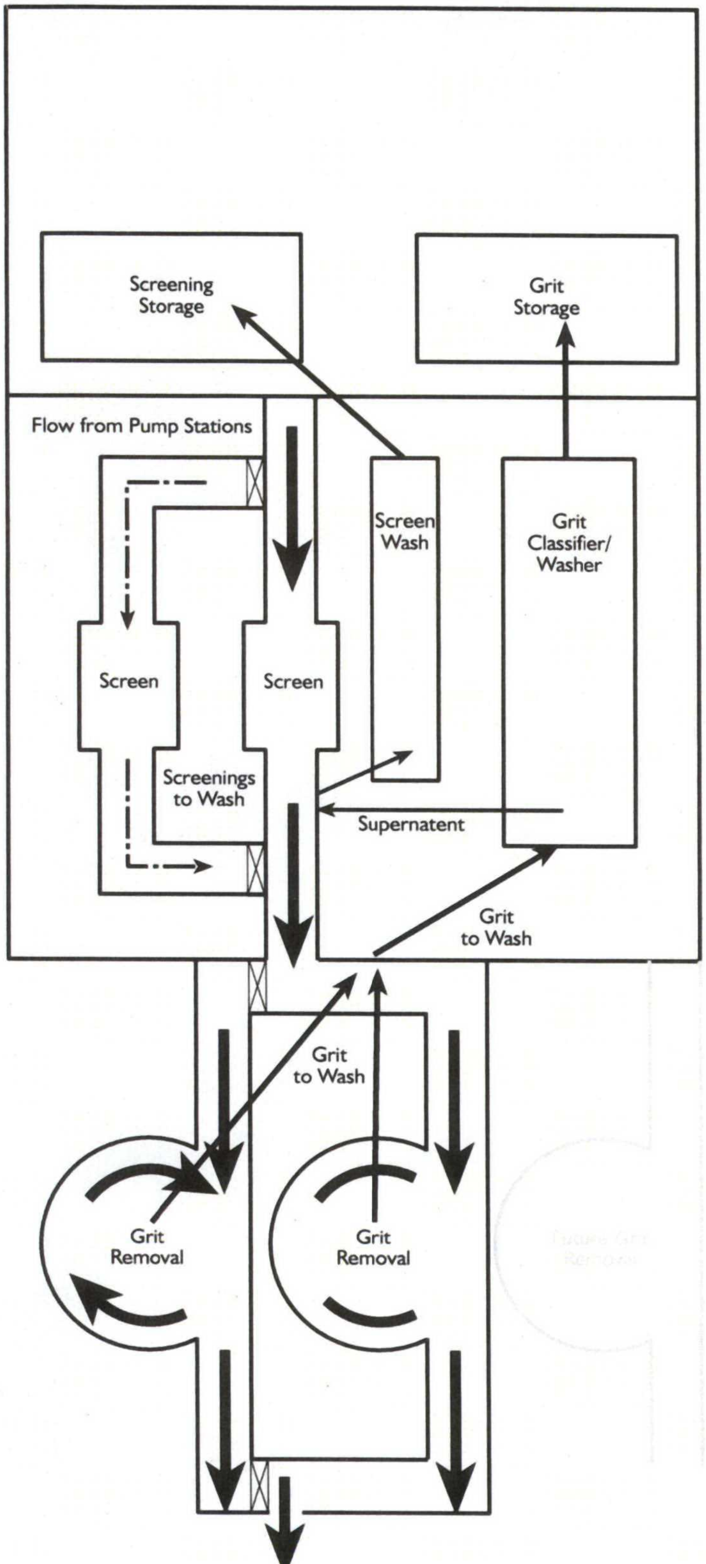
Primary Sludge and Scum Pumping

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.



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Project Title MOUNT VERNON COMPREHENSIVE SEWER	Date FEBRUARY 2003
Sheet Title PROPOSED HEADWORKS FACILITY	Figure No. 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.

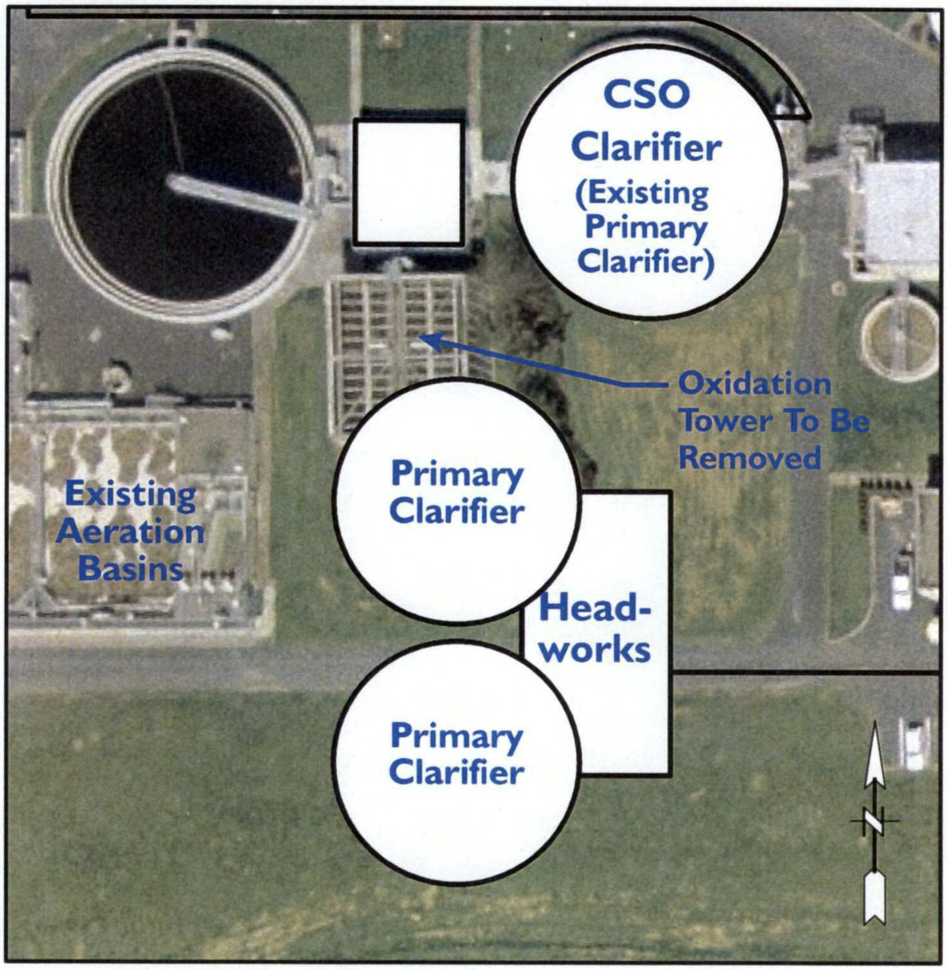
Cost Estimate

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.



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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.

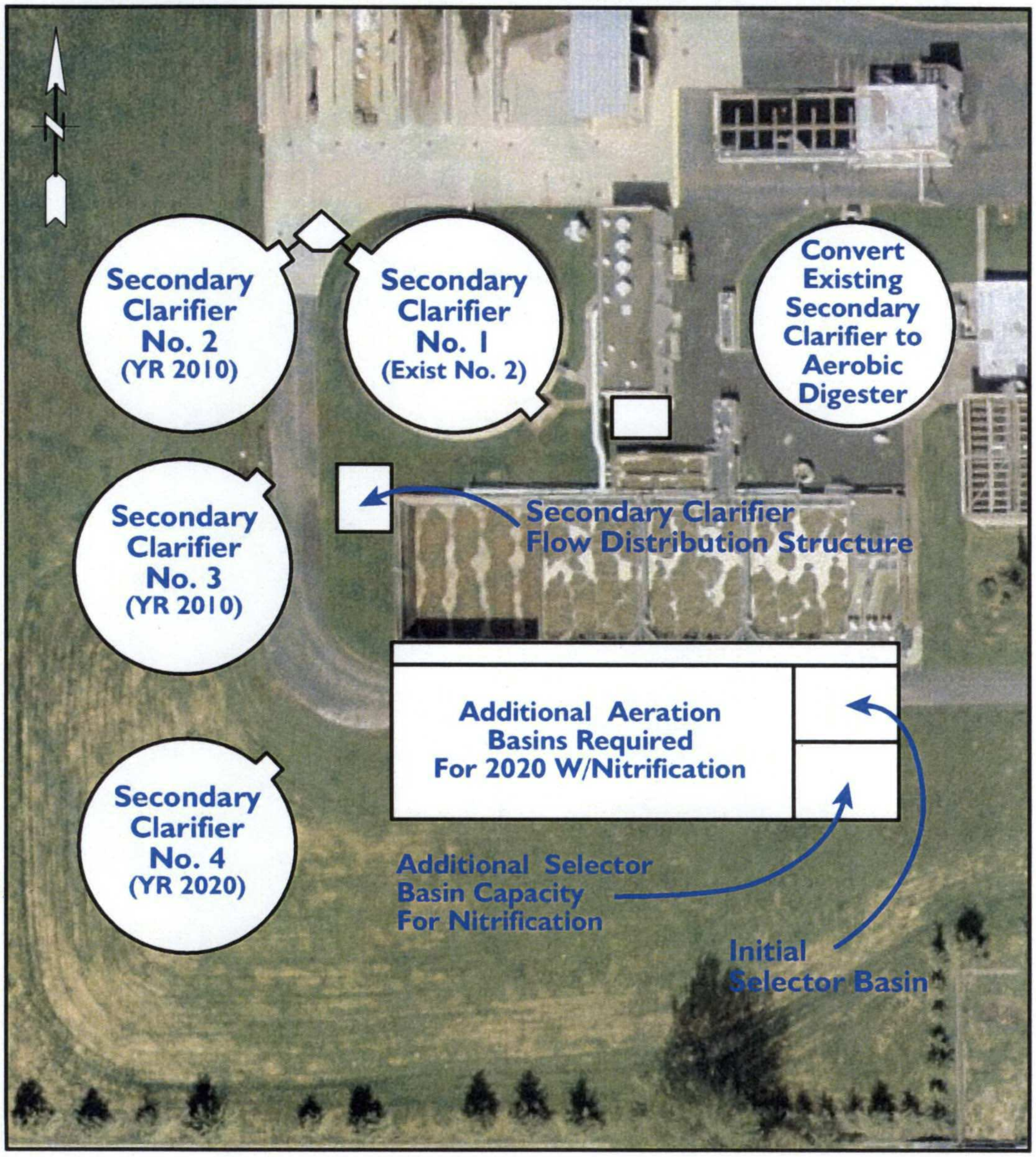
Aeration for the basins would be fine bubble, as explained in alternative A.

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



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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.

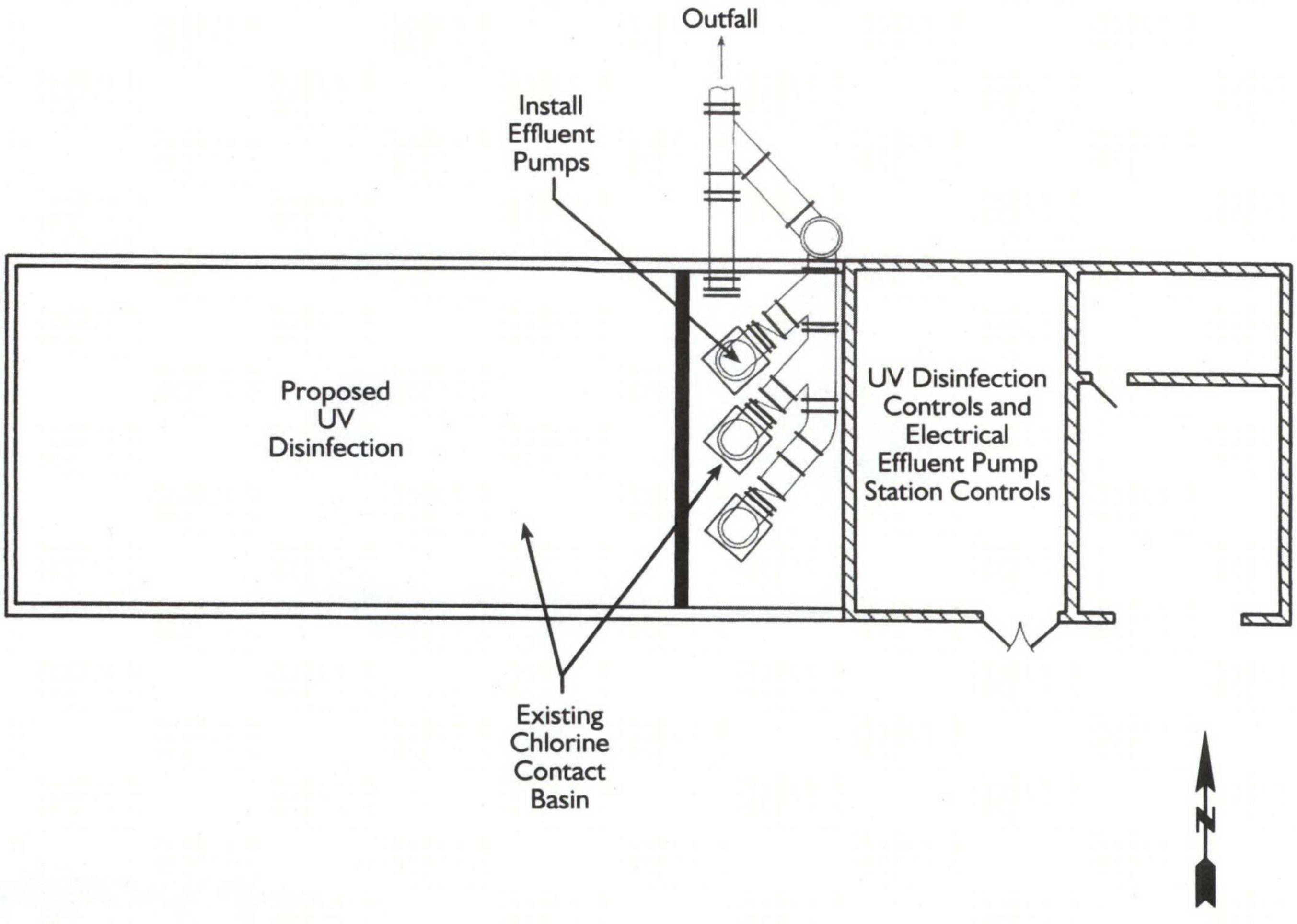
Alternative A - Retrofit Existing Effluent Pump Station

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 MOUNT VERNON COMPREHENSIVE SEWER	Date FEBRUARY 2003
Sheet Title UV DISINFECTION AND EFFLUENT PUMP STATION	Figure No. 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 • Single dilution zone	• Combined effluents would need to be addressed in future NPDES permits
Two Pipe	• Maintains option for separate CSO outfall • Lower maintenance (diffusers not required)	• Greater Capital Cost • 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 capacities 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			
Item	Co-generation Capacity		
	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.

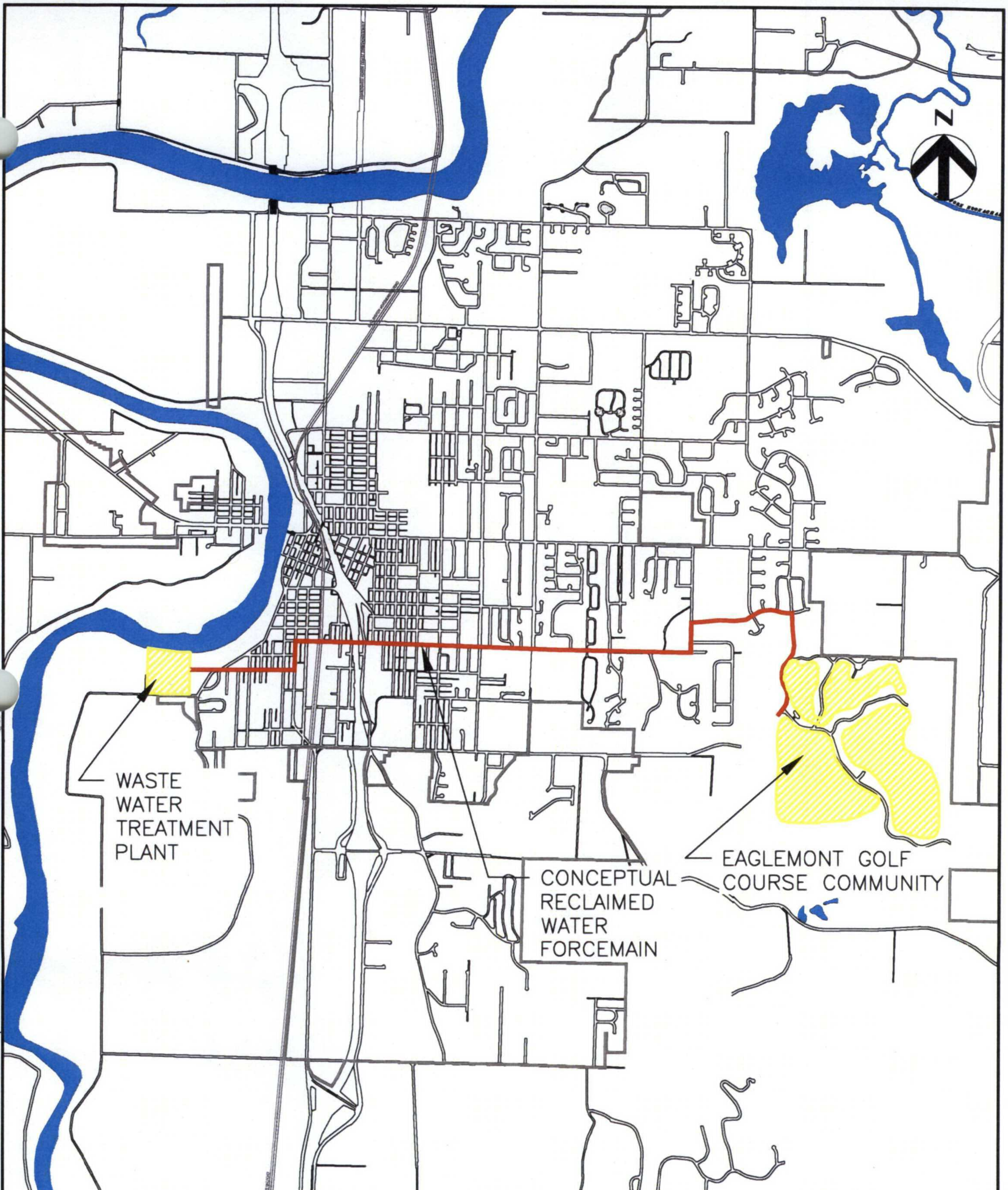
Table 9-13

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

DATE: 02/11/03

FILENAME: FIG9-10

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.

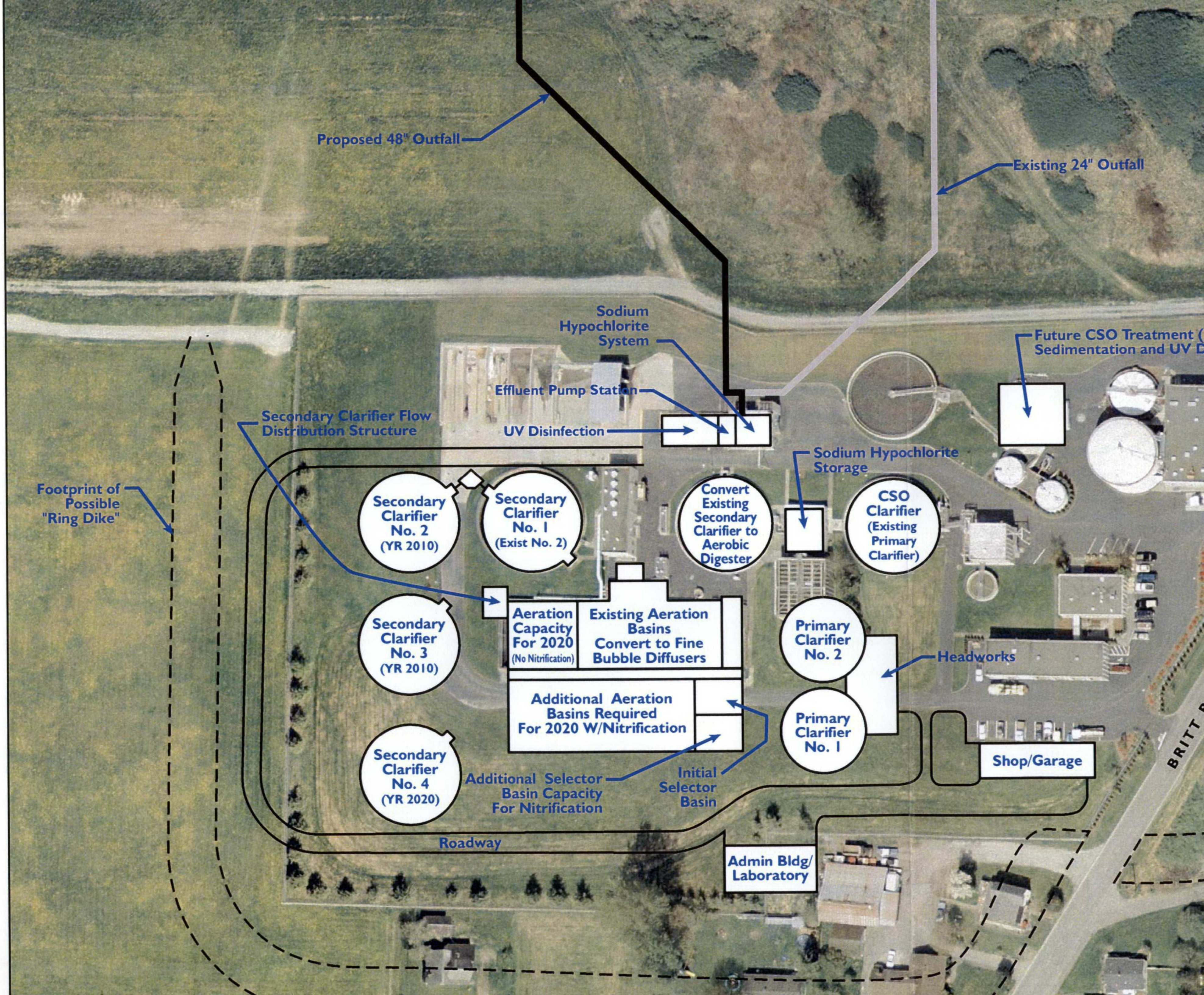
Summary of Recommendations

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.

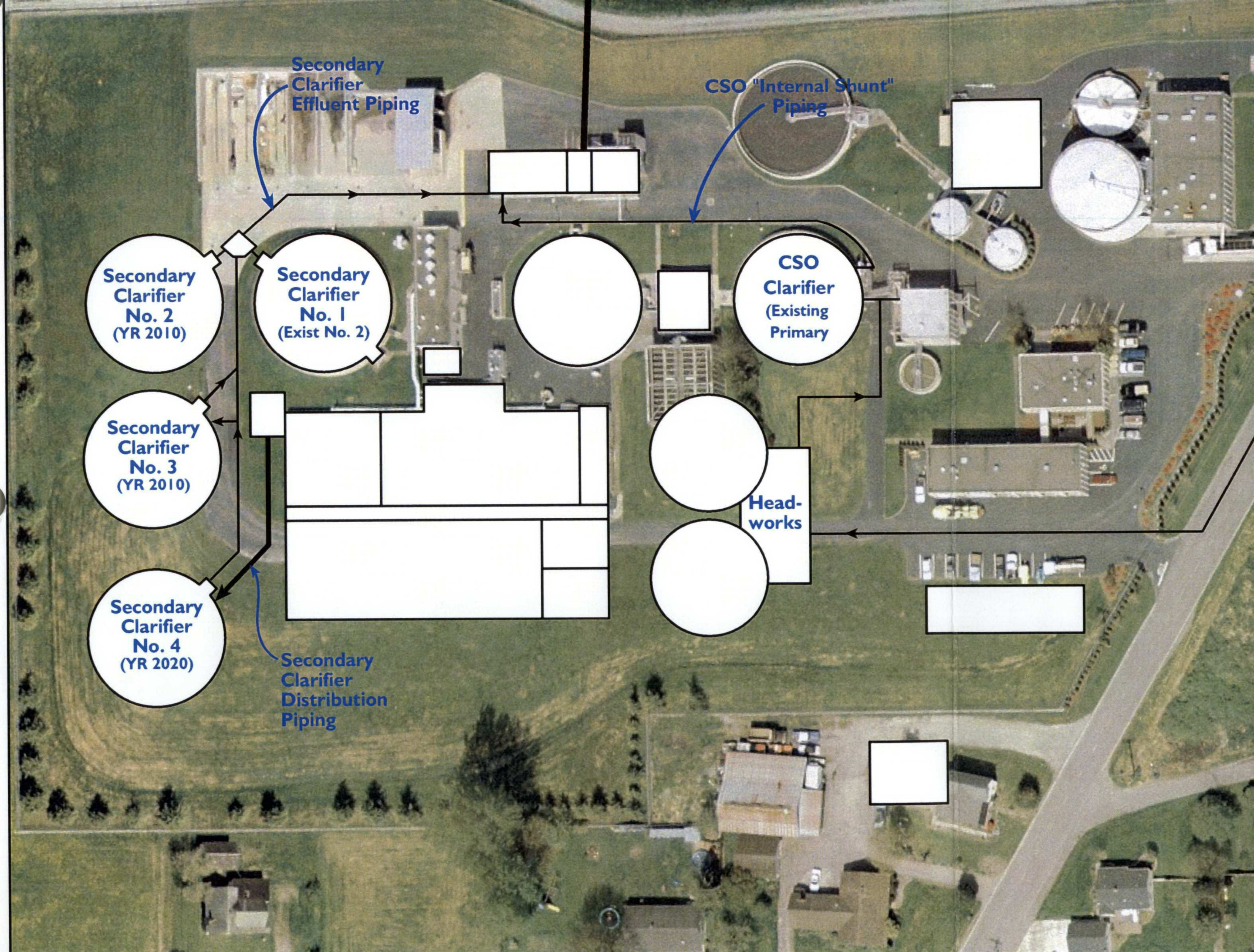
Table 10-1

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.



Proposed 48" Outfall
Existing 24" Outfall
Sodium Hypochlorite System
Future CSO Treatment (
Sedimentation and UV D
Effluent Pump Station
Secondary Clarifier Flow Distribution Structure
UV Disinfection
Sodium Hypochlorite Storage
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
CSO Clarifier (Existing Primary Clarifier)
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
Roadway
Admin Bldg/ Laboratory
BRITT


Secondary Clarifier Effluent Piping
CSO "Internal Shunt" Piping
Secondary Clarifier No. 2 (YR 2010)
Secondary Clarifier No. 1 (Exist No. 2)
CSO Clarifier (Existing Primary
Secondary Clarifier No. 3 (YR 2010)
Head-works
Secondary Clarifier No. 4 (YR 2020)
Secondary Clarifier Distribution Piping

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**	**201 1**	**201 2**	**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	201 1	201 2	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/Operation s 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	201 1	201 2	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	201 1	201 2	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	201 1	201 2	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 54				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			1,00 0												
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.

APPENDIX A
DRAPER VALLEY FARMS, INC.
20-YEAR FLOW PROJECTIONS

Exhibit B

DRAPER VALLEY
F•A•R•M•S

Ph: 360-424-7947 800-562-2012 Fx: 360-424-1666
CORPORATE OFFICES
P.O. Box 838 - 1000 Jason Lane, Mount Vernon, WA 98273

July 15, 1998

RECEIVED
CITY OF MT. VERNON

JUL 17 1998

WASTE WATER TREATMENT

Dear Mr. Dan Eisses,

RE: Load Projection for Draper Valley Farms over the next 20 years

Our projection could be as high as 140 birds per minute at five gallons a bird, fifteen hours a day, for a total of 630,000 gallons.

Sixty five million birds are processed in Washington and Oregon per year. Draper Valley is currently processing 22-25 million of those birds per year. We are capable of processing 30 million birds per year. As the population rises over the next twenty years, our production could go up as well. If this takes place, 630,000 gallons would escalate to 750,000 gallons.

The processing procedures are regulated by the USDA. These procedures include birds per day and the amount of water needed to process each bird. Therefore it is not always possible to project future uses and needs.

Sincerely,

John Jefferson

John Jefferson

APPENDIX B
L. A. PEAKING CURVE

SANITARY FLOW PEAKING FACTORS VS. DAILY FLOW CURVE

The peaking factor curve shown in Figure B-1 was developed by the Bureau of Engineering of the City of Los Angeles.[1]

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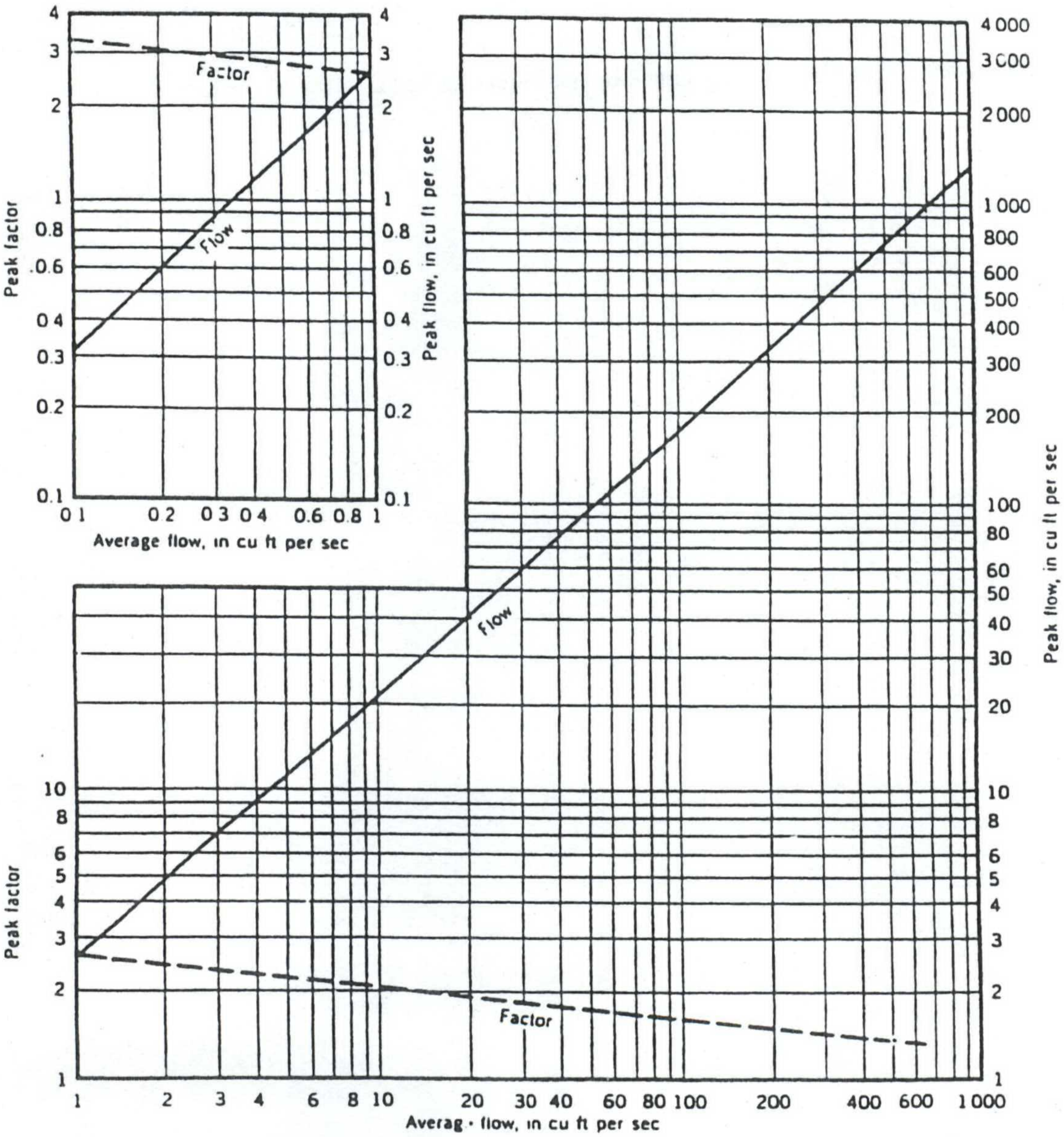
[1]City of Los Angeles, Bureau of Engineering. *ASCE-Manuals and Reports on Engineering Practice No. 37*, "Design and Construction of Sanitary and Storm Sewer." 1979.

QUANTITY OF SANITARY SEWAGE



Ratio of peak flow to average daily flow in Los Angeles.
(Cfs × 1.7 = cu m/min.)

Figure B-1
Sanitary Flow Peaking Factors vs. Daily Flow Curve

APPENDIX C
CITY OF MOUNT VERNON'S BASIN DELINEATION
FOR
HYDRAULIC MODELING

City of Mount Vernon
09637-005-002
3849

Basin Delineation	Area (ac)	Density (c/ac)	Infiltration Rate (gpad)	Contribution per capita (gpd)	Comment
N10-A, EAST END	42.5	6.04	1,100	100	
N10-J	208	8.9	1,100	100	
N10-G	34	8.9	1,100	100	
N10-H	40	8.9	1,100	100	
N10-I	40	8.9	1,100	100	
N10-D	34	8.9	1,100	100	
N10-E	40	8.9	1,100	100	
N10-F	40	8.9	1,100	100	
N10-B	40	8.9	1,100	100	
N10-C	40	8.9	1,100	100	
N11-C	71	3.55	1,100	100	
N16	119	8.91	1,100	100	
N11-Q	20	0	1,100	100	
N11-I	42	5.83	1,100	100	
N11-T	42.7	7.75	1,100	100	
N11-S	43.4	7.96	1,100	100	
N11-R	38	10.06	1,100	100	
N11-P	20	8.82	1,100	100	
N11-L	40	20.1	1,100	100	
N11-N	30	17.48	1,100	100	
N11-J	48.7	1.76	1,100	100	
N11-K	40	14.66	1,100	100	
N11-M	40	14.09	1,100	100	
N11-F	27.4	7.24	1,100	100	
N11-G	20	11.26	1,100	100	
N11-H	20	11.45	1,100	100	
N11-D	20	13.12	1,100	100	
N11-E	20	5.94	1,100	100	
N11-A	24	13.47	1,100	100	
N11-B	38	5.82	1,100	100	
N10-A, WEST END	20	6.04	1,100	100	
N13-F	61.2	6.33	1,100	100	
N13-C	89.7	4.8	1,100	100	
N13-A	48.7	5.03	1,100	100	
N13-E	49.4	6.4	1,100	100	
N13-D	20.7	6.3	1,100	100	
N13-B	92.7	4.6	1,100	100	
N12-H	52.9	8.9	1,100	100	
N12-G	54.6	8.3	1,100	100	
N12-F	51.2	5.7	1,100	100	
N12-E, PART	20	8.9	1,100	100	
N12-D, PART	14.9	8.9	1,100	100	
N12-E, PART	44.5	8.9	1,100	100	
N12-C	52.1	4	1,100	100	
N12-B	35.7	5.5	1,100	100	
SMALL SIDE AREA	8.8	6.4	1,100	100	
N12-A	30.5	10.3	1,100	100	
N14-H	71.1	9	1,100	100	
N14-G	46.5	6.6	1,100	100	
N14-E	8	10	1,100	100	
N14-C	22.7	8.9	1,100	100	
N14-E	24	10	1,100	100	

Basin Delineation	Area (ac)	Density (c/ac)	Infiltration Rate (gpad)	Contribution per capita (gpd)	Comment
N14-F	37.2	16.9	1,100	100	
N14-E	6	20	1,100	100	
N14-D	59.9	14.9	1,100	100	
N14-B	36.8	23.2	1,100	100	
N14-A	28.8	11.2	1,100	100	
N7-G	46.2	15.2	1,100	100	
N9-E, PART	42	3	1,100	100	
N15	370	8.9	1,100	100	
N9-F	20	8.9	1,100	100	
N9-D	78.2	8.9	1,100	100	
N9-C	30.5	7.2	1,100	100	
N9-A	117.1	8.1	1,100	100	
N9-B	41.8	17	1,100	100	
N11-O, COLLEGE	90	2.2	1,100	100	
N9-E, PART	43.2	15	1,100	100	
N7-C	38.7	8.9	1,100	100	
N7-A	25.2	23	1,100	100	
N7-B	69.4	14.5	1,100	100	
N7-F	51.3	7.1	1,100	100	
N7-D	41.3	8.6	1,100	100	
N7-E	33.1	8.6	1,100	100	
N8,	55.7	20.55	1,100	100	Commercial at 2,055 gpad
N8,	60	20.55	1,100	100	Commercial at 2,055 gpad
N6,	45.7	164.11	1,100	100	Draper valley at 0.75 mgd
N5-E	54.3	10.8	1,100	100	
N5-D	15.2	8.6	1,100	100	
N5-C	26.2	13.3	1,100	100	
N5-B	35.6	13.2	1,100	100	
N5-A,	43.4	13.2	1,100	100	
N4-A	8	13.3	1,100	100	
N4-B	45	16.06	1,100	100	
N4-C	45	13.3	1,100	100	
N4-D	27	9.2	1,100	100	
N2,	25	12.9	1,100	100	
N2,	10	0	1,100	100	
C2-C	119	6.05	1,100	100	
C2-B	65.9	4.46	1,100	100	
S5-D	640	3.1	1,100	100	
S5-A	302	8.9	1,100	100	
S5-B	203	9.1	1,100	100	
S5-C	97	8.8	1,100	100	
S4-A	120	8.9	1,100	100	
S4-B	154	8.9	1,100	100	
S4-C	280	8.8	1,100	100	
S3-A	134	8.9	1,100	100	
S3-B	70	8.9	1,100	100	
S3-C	151	8.9	1,100	100	
S1-D	138	8.9	1,100	100	
S1-B	137.2	7.14	1,100	100	
S2	104.2	5.9	1,100	100	
S1-C	26.4	5.6	1,100	100	
S1-A, EAST	48	9.6	1,100	100	
S1-A, WEST	84.3	6.2	1,100	100	
C3-B, SOUTH	29	6.08	1,100	100	
C3-B, NORTH	38.8	8.8	1,100	100	
C3-A, EAST	25.5	6.9	1,100	100	
C3-A, WEST	14.9	3.94	1,100	100	
N1-A	149.5	6.05	1,100	100	

Basin Delineation	Area (ac)	Density (c/ac)	Infiltration Rate (gpad)	Contribution per capita (gpd)	Comment
N1-B	109.7	6.11	1,100	100	
N1-C, TO DIVISION	19.7	7.6	1,100	100	
N3	151.2	14.1	1,100	100	
C1-A,	15	5.12	1,100	100	
C1-A,	71	16.6	1,100	100	
C2-A	70.4	3.82	1,100	100	
C1-D	30.1	4.9	1,100	100	
C1-B	28.9	18.4	1,100	100	
C1-C,	22	23.2	1,100	100	
C1-C,	44.7	4.4	1,100	100	
WEST MV, W1-A	108	5.7	1,100	100	
WEST MV, W1-B	220	8.9	1,100	100	
WEST MV, W2	200	8.6	1100	100	
SECTION 8/4	200	8.6	1100	100	
SECTION 23	560	8.6	1100	100	
SECTION 26	560	8.6	1100	100	

APPENDIX D
HYDRAULIC ANALYSIS OUTPUT OF THE CITY OF MOUNT VERNON'S WASTEWATER COLLECTION SYSTEM

City of Mount Vernon
09637-005-002
3849

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
	*****	COLLEGE	WAY	PUMP	STATIO	DRAIN	AREA									
	COLLEGE	WAY,	EAST	OF	PUMP	STATION										
N10-A, SEC 23, 26	90.00	84.00	72.53	71.57	175	8	0.0055	42.50	0.00	0.00	1.279	0.989	2.60	3.85	0.58	666.02
	83.50	76.50	71.57	69.80	99	8	0.0179	0.00	1.28	0.99	0.000	0.000	2.60	3.85	1.04	368.92
	76.50	61.00	69.80	55.00	160	8	0.0925	0.00	1.28	0.99	0.000	0.000	2.60	3.85	2.38	162.19
	61.00	58.50	55.00	50.60	270	8	0.0163	0.00	1.28	0.99	0.000	0.000	2.60	3.85	1.00	386.42
	58.50	47.00	50.60	37.40	270	8	0.0489	0.00	1.28	0.99	0.000	0.000	2.60	3.85	1.73	223.10
	47.00	40.00	37.40	29.44	405	8	0.0197	0.00	1.28	0.99	0.000	0.000	2.60	3.85	1.09	351.86
HOL	1								1.28	0.99						
UNDEVELOPED AREA	EAST	OF	TOWN,	SECTIONS	N10	B	THRU									
N10-J	90	84	392	386	1300	12	0.0046	208.00	0.00	0.00	0.229	0.185	3.04	0.79	1.56	50.59
N10-G,H,I	84	77	386	350	2600	12	0.0138	114.00	0.23	0.19	0.125	0.101	2.93	1.19	2.71	44.02
N10-D,E,F	77	61	350	225	1400	12	0.0893	114.00	0.35	0.29	0.125	0.101	2.85	1.58	6.88	23.02
N10-B,C	61	59	225	62	600	12	0.2717	80.00	0.48	0.39	0.088	0.071	2.80	1.85	12.00	15.45
	59	47	62	40	1900	12	0.0116	0.00	0.57	0.46	0.000	0.000	2.80	1.85	2.48	74.85
HOL	2								0.57	0.46						
	WAUGH	RD	SOUTH													
N11-C,D,A	66.00	59.00	55.00	48.54	322	8	0.0201	115.00	0.00	0.00	0.127	0.084	3.25	0.40	1.11	36.02
	59.00	54.50	48.54	43.80	316	8	0.015	0.00	0.13	0.08	0.000	0.000	3.25	0.40	0.96	41.65
	54.50	52.50	43.80	42.38	284	8	0.005	0.00	0.13	0.08	0.000	0.000	3.25	0.40	0.55	72.15
	52.50	52.50	42.38	41.14	248	8	0.005	0.00	0.13	0.08	0.000	0.000	3.25	0.40	0.55	72.15
	52.50	50.00	41.14	38.61	211	8	0.012	0.00	0.13	0.08	0.000	0.000	3.25	0.40	0.86	46.59
HOL	3								0.13	0.08						
	WAUGH	RD	NORTH													
N16, N11-Q, N11-I	61.00	53.00	50.60	44.12	432	8	0.015	181.00	0.00	0.00	0.199	0.131	3.13	0.61	0.96	63.53
	53.00	51.00	44.12	41.71	344	8	0.007	0.00	0.20	0.13	0.000	0.000	3.13	0.61	0.65	92.96
	51.00	46.50	41.71	39.78	287	8	0.0067	0.00	0.20	0.13	0.000	0.000	3.13	0.61	0.64	94.88
	46.50	47.50	39.70	38.98	103	8	0.007	0.00	0.20	0.13	0.000	0.000	3.13	0.61	0.65	93.06
	47.50	50.00	38.98	38.61	247	12	0.0015	0.00	0.20	0.13	0.000	0.000	3.13	0.61	0.89	68.19
HOL	4								0.20	0.13						
	30TH	ROW														
N11-T	147.00	130.00	139.93	122.93	1000	8	0.017	42.7	0.00	0.00	0.047	0.033	3.49	0.16	1.02	15.95
	130.00	125.00	122.93	118.15	281	8	0.017	0.00	0.05	0.03	0.000	0.000	3.49	0.16	1.02	15.94
	125.00	115.00	118.15	110.13	277	8	0.029	0.00	0.05	0.03	0.000	0.000	3.49	0.16	1.33	12.22
	115.00	106.00	110.13	104.74	270	8	0.02	0.00	0.05	0.03	0.000	0.000	3.49	0.16	1.10	14.72
N11-S,R,P	106.00	80.00	104.74	60.00	2700	8	0.0166	101.40	0.05	0.03	0.112	0.090	3.14	0.55	1.01	54.40
HOL	5								0.16	0.12						
	COLLEGE	WAY,	26TH	TO	WAUGH	RD										

City of Mount Vernon Comprehensive Sewer Plan Update

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
	COLLEGE	WAY,	WAUGH	RD TO	PUMP	STA										
	COLLEGE	WAY	PUMP	STATION												
N11-L,N	79.00	73.50	69.57	65.00	330	8	0.0138	70.00	0.00	0.00	0.077	0.133	3.13	0.49	0.92	53.56
	73.50	71.50	65.00	62.34	380	8	0.007	0.00	0.08	0.13	0.000	0.000	3.13	0.49	0.65	75.33
	71.50	67.50	62.34	59.67	380	10	0.007	0.00	0.08	0.13	0.000	0.000	3.13	0.49	1.19	41.47
REC 5	67.50	65.50	59.67	56.67	428	10	0.007	0.00	0.24	0.26	0.000	0.000	2.95	0.99	1.19	83.75
	65.50	61.00	56.67	52.77	390	12	0.01	0.00	0.24	0.26	0.000	0.000	2.95	0.99	2.30	43.12
	61.00	58.00	52.77	48.87	390	12	0.01	0.00	0.24	0.26	0.000	0.000	2.95	0.99	2.30	43.12
N11-F,G,H	58.00	54.50	48.87	46.10	302	12	0.0092	67.40	0.24	0.26	0.074	0.065	2.90	1.24	2.21	56.27
N11-J,K	54.50	54.00	46.10	45.30	88	12	0.0091	88.70	0.31	0.32	0.098	0.067	2.85	1.51	2.20	68.95
N11-M	54.00	50.00	45.30	41.90	400	12	0.0085	40.00	0.41	0.39	0.044	0.056	2.81	1.70	2.12	80.20
N11-E	50.00	51.50	41.90	40.29	268	12	0.006	20.00	0.45	0.45	0.022	0.012	2.80	1.75	1.78	98.32
	51.50	50.00	40.29	38.61	280	12	0.006	0.00	0.47	0.46	0.000	0.000	2.80	1.75	1.78	98.38
REC 3,4 & N11-B	50.00	48.00	38.61	37.17	360	8	0.004	38.00	0.80	0.67	0.042	0.022	2.70	2.71	0.49	548.58
	50.00	45.00	37.17	35.79	345	8	0.004	0.00	0.84	0.69	0.000	0.000	2.70	2.71	0.49	548.58
	45.00	43.00	35.79	34.39	350	8	0.004	0.00	0.84	0.69	0.000	0.000	2.70	2.71	0.49	548.58
	43.00	43.50	34.39	32.99	350	8	0.004	0.00	0.84	0.69	0.000	0.000	2.70	2.71	0.49	548.58
N10-A, WEST END	43.50	42.00	32.99	31.59	350	8	0.004	20	0.84	0.69	0.022	0.012	2.69	2.76	0.49	558.98
	42.00	39.50	31.59	30.53	265	8	0.004	0.00	0.86	0.71	0.000	0.000	2.69	2.76	0.49	558.98
	39.50	40.00	30.53	29.44	272	8	0.004	0.00	0.86	0.71	0.000	0.000	2.69	2.76	0.49	558.47
REC 1,2	40.00	40.00	29.44	29.29	15	8	0.01	0.00	2.71	2.15	0.000	0.000	2.40	7.88	0.78	1008.68
HOL	6	----------------	FLOWS	FROM	COLLEGE	WAY	PUMP	1120.00	2.71	2.15						
	***********	DRAINAGE	AREA	N13	********************************											
	WAUGH	RD	AND	RIDGE	WAY											
	FIR	ST.,	WAUGH	TO	COMANCHE											
N13-F	384.00	374.00	374.03	364.64	220	8	0.0427	61.2	0.00	0.00	0.067	0.039	3.45	0.20	1.61	12.45
	374.00	358.00	364.64	349.14	345	8	0.0449	0.00	0.07	0.04	0.000	0.000	3.45	0.20	1.66	12.13
	358.00	356.00	349.14	347.82	330	8	0.004	0.00	0.07	0.04	0.000	0.000	3.45	0.20	0.49	40.66
	356.00	354.00	347.82	346.17	415	8	0.004	0.00	0.07	0.04	0.000	0.000	3.45	0.20	0.49	40.79
	354.00	341.00	346.17	333.50	300	8	0.0422	0.00	0.07	0.04	0.000	0.000	3.45	0.20	1.61	12.51
	341.00	335.00	333.50	327.10	290	8	0.0221	0.00	0.07	0.04	0.000	0.000	3.45	0.20	1.16	17.31
	335.00	331.00	327.10	324.88	250	8	0.0089	0.00	0.07	0.04	0.000	0.000	3.45	0.20	0.74	27.29
	331.00	308.00	324.88	299.42	128	8	0.1989	0.00	0.07	0.04	0.000	0.000	3.45	0.20	3.48	5.77
	308.00	290.00	299.42	282.22	312	8	0.0551	0.00	0.07	0.04	0.000	0.000	3.45	0.20	1.83	10.95
	290.00	275.00	282.22	268.15	335	8	0.042	0.00	0.07	0.04	0.000	0.000	3.45	0.20	1.60	12.55
	275.00	211.00	268.15	200.40	485	8	0.1397	0.00	0.07	0.04	0.000	0.000	3.45	0.20	2.92	6.88
N13-C	211.00	193.00	200.40	184.95	125	8	0.1236	89.7	0.07	0.04	0.099	0.043	3.25	0.43	2.75	15.73
	193.00	180.00	184.95	175.03	216	8	0.0459	0.00	0.17	0.08	0.000	0.000	3.25	0.43	1.67	25.81
	180.00	162.00	175.03	154.00	363	8	0.0579	0.00	0.17	0.08	0.000	0.000	3.25	0.43	1.88	22.98
	162.00	130.00	154.00	124.00	400	8	0.075	0.00	0.17	0.08	0.000	0.000	3.25	0.43	2.14	20.20
	130.00	122.00	124.00	113.18	135	8	0.0801	0.00	0.17	0.08	0.000	0.000	3.25	0.43	2.21	19.54
	122.00	97.00	113.18	86.50	334	12	0.0799	0.00	0.17	0.08	0.000	0.000	3.25	0.43	6.51	6.64
N13-A	97.00	94.00	86.50	85.38	460	12	0.0024	48.7	0.17	0.08	0.054	0.024	3.18	0.56	1.14	49.11
HOL	7								0.22	0.11						
	DIVISION,	37TH	TO	SIOUX	DR.											
	SIOUX,	SHOSHONE	TO	FIR	ST											
	FIR	ST.,COMAN	TO	30TH												
N13-E	356.00	352.00	345.70	341.60	82	8	0.05	49.4	0.00	0.00	0.054	0.032	3.50	0.16	1.75	9.45
	352.00	311.00	341.60	300.48	380	8	0.1082	0.00	0.05	0.03	0.000	0.000	3.50	0.16	2.57	6.42

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
N13-D	311.00	297.00	300.48	283.10	378	8	0.046	20.7	0.05	0.03	0.023	0.013	3.41	0.23	1.67	13.70
	297.00	287.00	283.10	276.45	145	8	0.0459	0.00	0.08	0.04	0.000	0.000	3.41	0.23	1.67	13.71
	287.00	276.00	276.45	269.45	350	8	0.02	0.00	0.08	0.04	0.000	0.000	3.41	0.23	1.10	20.77
	276.00	254.00	269.45	244.95	350	8	0.07	0.00	0.08	0.04	0.000	0.000	3.41	0.23	2.07	11.10
	254.00	247.00	244.95	238.05	220	8	0.0314	0.00	0.08	0.04	0.000	0.000	3.41	0.23	1.38	16.58
	247.00	242.00	238.05	235.08	120	8	0.0248	0.00	0.08	0.04	0.000	0.000	3.41	0.23	1.23	18.67
	242.00	222.00	235.08	213.42	430	8	0.0504	0.00	0.08	0.04	0.000	0.000	3.41	0.23	1.75	13.09
	222.00	208.00	213.42	201.96	200	8	0.0573	0.00	0.08	0.04	0.000	0.000	3.41	0.23	1.87	12.27
	208.00	184.00	201.96	171.26	270	8	0.1137	0.00	0.08	0.04	0.000	0.000	3.41	0.23	2.63	8.71
N13-B	184.00	170.00	171.26	162.22	460	8	0.0197	92.7	0.08	0.04	0.102	0.043	3.24	0.46	1.09	42.15
	170.00	163.00	162.22	153.20	140	8	0.0644	0.00	0.18	0.09	0.000	0.000	3.24	0.46	1.98	23.28
	163.00	136.00	153.20	127.45	250	8	0.103	0.00	0.18	0.09	0.000	0.000	3.24	0.46	2.51	18.41
	136.00	130.00	127.45	118.96	65	8	0.1306	0.00	0.18	0.09	0.000	0.000	3.24	0.46	2.82	16.35
	130.00	108.00	118.96	98.16	150	8	0.1387	0.00	0.18	0.09	0.000	0.000	3.24	0.46	2.91	15.87
	108.00	94.00	98.16	85.38	240	8	0.0533	0.00	0.18	0.09	0.000	0.000	3.24	0.46	1.80	25.61
REC 7	94.00	94.00	85.38	84.73	320	12	0.002	0.00	0.40	0.19	0.000	0.000	3.03	0.98	1.04	94.89
	94.00	94.00	84.73	84.08	330	12	0.002	0.00	0.40	0.19	0.000	0.000	3.03	0.98	1.02	96.36
	94.00	94.00	84.08	83.43	330	12	0.002	0.00	0.40	0.19	0.000	0.000	3.03	0.98	1.02	96.36
HOL	8								0.40	0.19						
	***********	DRAINAGE	AREA	N12	*********************************											
	DIVISION,	ABOUT	32ND	TO	MANITO											
	MANITO	DR.,	DIVISION	TO	THE	NORTH										
N12-H,G,F	231.00	230.00	220.61	218.85	290	8	0.0061	158.70	0.00	0.00	0.175	0.122	3.15	0.56	0.61	91.62
	230.00	225.00	218.85	216.00	280	8	0.0102	0.00	0.17	0.12	0.000	0.000	3.15	0.56	0.79	70.75
	225.00	219.00	216.00	208.09	340	8	0.0233	0.00	0.17	0.12	0.000	0.000	3.15	0.56	1.19	46.79
	219.00	212.00	208.09	203.01	140	8	0.0363	0.00	0.17	0.12	0.000	0.000	3.15	0.56	1.49	37.47
	212.00	205.00	203.01	198.89	405	8	0.0102	0.00	0.17	0.12	0.000	0.000	3.15	0.56	0.79	70.77
	205.00	192.00	198.89	181.91	310	8	0.0548	0.00	0.17	0.12	0.000	0.000	3.15	0.56	1.83	30.50
	192.00	175.00	181.91	161.78	340	8	0.0592	0.00	0.17	0.12	0.000	0.000	3.15	0.56	1.90	29.33
N12-E	175.00	141.00	161.78	128.01	320	8	0.1055	20	0.17	0.12	0.022	0.018	3.11	0.63	2.54	24.85
	141.00	132.00	128.01	118.02	320	8	0.0312	0.00	0.20	0.14	0.000	0.000	3.11	0.63	1.38	45.69
N12-D	132.00	132.00	118.02	115.93	530	8	0.0039	14.9	0.20	0.14	0.016	0.013	3.09	0.68	0.49	139.59
HOL	9								0.21	0.15						
	30TH	ST.,	TO	FIR	ST.											
	FIR	ST.,	30TH	TO	26TH											
	26TH,	FIR	TO	KULSHAN												
N12-E	163.00	132.00	155.45	115.93	380	8	0.104	44.5	0.00	0.00	0.049	0.040	3.44	0.19	2.52	7.35
REC 9, N12-C	132.00	107.00	115.93	91.93	400	8	0.06	52.1	0.26	0.19	0.057	0.021	3.00	0.96	1.91	50.13
	107.00	94.00	91.93	83.43	400	8	0.0213	0.00	0.32	0.21	0.000	0.000	3.00	0.96	1.14	84.24
REC 8	94.00	94.00	83.43	82.10	665	12	0.002	0.00	0.72	0.41	0.000	0.000	2.83	1.87	1.03	181.64
	94.00	92.00	82.10	81.26	420	12	0.002	0.00	0.72	0.41	0.000	0.000	2.83	1.87	1.03	181.64
N12-B	92.00	92.00	81.26	79.78	180	12	0.0082	35.7	0.72	0.41	0.039	0.020	2.82	1.96	2.09	93.88
	92.00	87.00	79.78	77.52	205	12	0.011	0.00	0.76	0.43	0.000	0.000	2.82	1.96	2.42	81.08
SM SD AREA	87.00	83.00	77.52	72.20	490	12	0.0109	8.8	0.76	0.43	0.010	0.006	2.82	1.98	2.40	82.70
	83.00	78.00	72.20	68.75	690	12	0.005	0.00	0.77	0.43	0.000	0.000	2.82	1.98	1.63	121.87
HOL	10	----------------	SAVE	FIR	ST.	FLOWS			0.77	0.43						
	PARK	VILLAGE	DEVELOPMENT													0.00
N12-A	77.00	78.00	69.58	68.89	230	8	0.003	30.5	0.00	0.00	0.034	0.031	3.50	0.14	0.43	33.56

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
	78.00	78.00	68.89	68.75	70	8	0.002	0.00	0.03	0.03	0.000	0.000	3.50	0.14	0.35	41.10
HOL	11	----------------	SAVE	PARK	VILLAGE	FLOWS			0.03	0.03						
	***********	COMBINATI	OF	AREAS	N11,	N12	AND	N13								
	26TH,	COLLEGE	TO	KULSHAN												
	KULSHAN	ROW,	26TH	TO	LAVENTURE											
REC 6	83.00	80.00	74.17	72.86	330	12	0.004	0.00	2.71	2.15	0.000	0.000	2.40	7.88	1.45	543.00
	80.00	78.00	72.86	71.60	315	8	0.004	0.00	2.71	2.15	0.000	0.000	2.40	7.88	0.49	1594.86
	78.00	77.00	71.60	70.34	315	8	0.004	0.00	2.71	2.15	0.000	0.000	2.40	7.88	0.49	1594.86
	77.00	78.00	70.34	69.14	300	12	0.004	0.00	2.71	2.15	0.000	0.000	2.40	7.88	1.46	540.93
	78.00	78.00	69.14	68.75	25	12	0.0156	0.00	2.71	2.15	0.000	0.000	2.40	7.88	2.88	273.91
	78.00	79.00	68.75	67.44	330	12	0.004	0.00	2.71	2.15	0.000	0.000	2.40	7.88	1.45	543.00
	79.00	79.00	67.44	66.12	330	12	0.004	0.00	2.71	2.15	0.000	0.000	2.40	7.88	1.46	540.93
	79.00	78.00	66.12	64.81	330	12	0.004	0.00	2.71	2.15	0.000	0.000	2.40	7.88	1.45	543.00
	78.00	76.00	64.81	63.50	330	12	0.004	0.00	2.71	2.15	0.000	0.000	2.40	7.88	1.45	543.00
HOL	12	<--------	HOLD	ALL	FLOWS	FROM	KULSHA	WAY	2.71	2.15						
	***********	DRAINAGE	AREA	N14	********************************											
	DIVISION	ST.,	TRAILOR	PARK	TO	25TH										
N14-H,G	200.00	190.00	189.02	182.11	280	10	0.0247	117.60	0.00	0.00	0.129	0.095	3.21	0.43	2.22	19.50
N14-E	190.00	187.00	182.11	176.61	360	10	0.0153	8	0.13	0.09	0.009	0.008	3.19	0.47	1.75	26.63
N14-C	187.00	187.00	176.62	176.33	60	10	0.0048	22.7	0.14	0.10	0.025	0.020	3.15	0.55	0.98	55.84
N14-E	187.00	188.00	176.33	175.10	310	10	0.004	24	0.16	0.12	0.026	0.024	3.10	0.64	0.89	72.29
	188.00	189.00	175.10	173.93	295	10	0.004	0.00	0.19	0.15	0.000	0.000	3.10	0.64	0.89	72.31
	189.00	189.00	173.93	173.00	60	10	0.0155	0.00	0.19	0.15	0.000	0.000	3.10	0.64	1.76	36.58
HOL	13								0.19	0.15						
	LAVENTUR	KINCAID	TO	DIVISION												
	LAVENTUR	DIVISION	TO	FIR												
	LAVENTUR	FIR	TO	KULSHAN												
N14-F	203.00	200.00	192.48	191.43	225	8	0.0047	37.2	0.00	0.00	0.041	0.063	3.32	0.25	0.53	46.80
N14-E	200.00	199.00	191.43	190.00	285	8	0.005	6	0.04	0.06	0.007	0.012	3.28	0.29	0.55	52.92
	199.00	195.00	190.00	186.00	400	8	0.01	0.00	0.05	0.07	0.000	0.000	3.28	0.29	0.78	37.48
	195.00	190.00	186.00	178.10	395	8	0.02	0.00	0.05	0.07	0.000	0.000	3.28	0.29	1.10	26.50
	190.00	190.00	178.10	173.00	60	8	0.085	0.00	0.05	0.07	0.000	0.000	3.28	0.29	2.28	12.86
REC 13 & N14-D	190.00	178.00	173.00	165.87	325	8	0.0219	59.9	0.24	0.22	0.066	0.089	2.90	1.21	1.16	104.27
	178.00	171.00	165.87	159.25	300	8	0.0221	0.00	0.30	0.31	0.000	0.000	2.90	1.21	1.16	103.97
	171.00	156.00	159.25	146.00	600	8	0.0221	0.00	0.30	0.31	0.000	0.000	2.90	1.21	1.16	103.93
	156.00	155.00	146.00	143.00	113	8	0.0265	0.00	0.30	0.31	0.000	0.000	2.90	1.21	1.27	94.78
N14-B	155.00	148.00	143.00	138.00	187	8	0.0267	36.8	0.30	0.31	0.040	0.085	2.84	1.47	1.28	115.07
	148.00	108.00	138.00	95.00	657	8	0.0654	0.00	0.34	0.40	0.000	0.000	2.84	1.47	2.00	73.55
N14-A	108.00	97.00	95.00	84.00	495	8	0.0222	28.8	0.34	0.40	0.032	0.032	2.82	1.58	1.16	136.07
N7-G	97.00	89.00	84.00	77.77	366	10	0.017	46.2	0.38	0.43	0.051	0.070	2.78	1.81	1.85	98.15
N9-E	89.00	78.00	77.77	65.50	720	10	0.017	42	0.43	0.50	0.046	0.013	2.77	1.89	1.85	102.31
	78.00	66.00	65.50	63.40	300	10	0.007	0.00	0.47	0.51	0.000	0.000	2.77	1.89	1.18	159.63
HOL	14								0.47	0.51						
	***********	DRAINAGE	BASIN	N9	******************************											
	FRANCIS	ROAD														
	N.	LAVENTURE	TO	HOAG	ROAD											
	N.	LAVENTURE	HOAG	RD	TO	COLLEGE										

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
	LAVENTUR	COLLEGE	TO	KULSHAN												
N15, N9-F,D	130.00	122.00	124.10	115.10	200	12	0.0058	468.20	0.00	0.00	0.515	0.417	2.83	1.69	1.75	96.57
	122.00	113.00	115.10	105.84	300	12	0.0111	0.00	0.52	0.42	0.000	0.000	2.83	1.69	2.43	69.80
	113.00	105.00	105.84	98.70	350	12	0.004	0.00	0.52	0.42	0.000	0.000	2.83	1.69	1.46	116.28
	105.00	92.00	98.78	88.97	300	12	0.004	0.00	0.52	0.42	0.000	0.000	2.83	1.69	1.46	116.28
	130.00	122.00	124.10	115.10	348	12	0.004	0.00	0.52	0.42	0.000	0.000	2.83	1.69	1.46	116.28
	122.00	113.00	115.10	105.84	350	12	0.0048	0.00	0.52	0.42	0.000	0.000	2.83	1.69	1.60	106.15
	113.00	105.00	105.84	98.70	168	12	0.0218	0.00	0.52	0.42	0.000	0.000	2.83	1.69	3.40	49.81
	105.00	92.00	98.78	88.97	158	12	0.0125	0.00	0.52	0.42	0.000	0.000	2.83	1.69	2.57	65.78
N9-C	92.00	96.00	88.97	88.17	440	15	0.008	30.5	0.52	0.42	0.034	0.022	2.81	1.78	3.73	47.75
	96.00	97.00	88.17	86.22	395	15	0.008	0.00	0.55	0.44	0.000	0.000	2.81	1.78	3.73	47.75
N9-A, SEC 4	97.00	98.00	86.22	85.32	320	15	0.008	317.1	0.55	0.44	0.349	0.267	2.69	2.80	3.73	74.87
	98.00	97.00	85.22	84.66	330	15	0.008	0.00	0.90	0.71	0.000	0.000	2.69	2.80	3.73	74.87
N9-B	97.00	98.00	84.66	84.00	320	15	0.008	41.8	0.90	0.71	0.046	0.071	2.67	3.01	3.73	80.70
	98.00	99.00	84.00	83.34	320	15	0.008	0.00	0.94	0.78	0.000	0.000	2.67	3.01	3.73	80.70
	99.00	97.00	83.34	82.68	320	15	0.008	0.00	0.94	0.78	0.000	0.000	2.67	3.01	3.73	80.70
	97.00	94.00	82.68	82.02	320	15	0.008	0.00	0.94	0.78	0.000	0.000	2.67	3.01	3.73	80.70
	94.00	90.00	82.02	79.38	320	15	0.008	0.00	0.94	0.78	0.000	0.000	2.67	3.01	3.73	80.70
N11-O	90.00	86.00	79.38	73.86	320	15	0.008	90	0.94	0.78	0.099	0.020	2.66	3.16	3.73	84.63
	86.40	81.00	74.30	71.70	329.52	18	0.0079	0.00	1.04	0.80	0.000	0.000	2.66	3.16	6.03	52.40
N9-E	81.00	77.20	71.70	68.90	328.5	18	0.0085	43.2	1.04	0.80	0.048	0.065	2.64	3.36	6.27	53.65
	77.20	75.80	68.90	66.30	331.56	18	0.0078	0.00	1.09	0.86	0.000	0.000	2.64	3.36	6.01	55.93
	75.80	76.60	66.30	63.60	319.65	18	0.0084	0.00	1.09	0.86	0.000	0.000	2.64	3.36	6.24	53.89
	76.60	76.74	63.60	63.35	26.2	18	0.0095	0.00	1.09	0.86	0.000	0.000	2.64	3.36	6.63	50.70
HOL	15								1.09	0.86						
	***********	KULSHAN	INTERCEPTOR	******************************												
	KULSHAN	ROW	INTERCEPTOR													
REC 10,11	76.50	78.87	70.36	68.95	350	27	0.004	0.00	0.80	0.46	0.000	0.000	2.80	2.10	12.70	16.51
	78.87	78.24	68.95	68.55	100.5	27	0.004	0.00	0.80	0.46	0.000	0.000	2.80	2.10	12.63	16.61
	78.24	79.20	68.55	67.12	356.38	27	0.004	0.00	0.80	0.46	0.000	0.000	2.80	2.10	12.68	16.55
	79.20	77.59	67.12	65.92	300	27	0.004	0.00	0.80	0.46	0.000	0.000	2.80	2.10	12.66	16.57
	77.59	77.30	65.92	64.72	300	27	0.004	0.00	0.80	0.46	0.000	0.000	2.80	2.10	12.66	16.57
	77.30	76.50	64.72	63.52	300	27	0.004	0.00	0.80	0.46	0.000	0.000	2.80	2.10	12.66	16.57
	76.50	76.74	63.52	63.35	41.27	27	0.0041	0.00	0.80	0.46	0.000	0.000	2.80	2.10	12.85	16.33
	76.74	69.63	63.35	60.47	240.28	24	0.012	0.00	0.80	0.46	0.000	0.000	2.80	2.10	16.01	13.11
N7-C, REC 12, 14, 15	69.63	69.14	60.47	58.40	57.55	24	0.036	38.7	5.07	3.99	0.043	0.034	2.24	14.12	27.73	50.91
N7-F	69.14	68.44	58.40	55.97	67.45	24	0.036	51.3	5.11	4.02	0.056	0.036	2.23	14.24	27.75	51.33
	68.44	67.32	55.97	55.64	83.25	24	0.004	0.00	5.17	4.06	0.000	0.000	2.23	14.24	9.21	154.74
	67.32	61.50	55.64	53.83	453.2	24	0.004	0.00	5.17	4.06	0.000	0.000	2.23	14.24	9.24	154.16
	61.50	61.50	53.83	52.36	367.74	24	0.004	0.00	5.17	4.06	0.000	0.000	2.23	14.24	9.24	154.10
	61.50	65.00	52.36	52.21	38.85	24	0.0039	0.00	5.17	4.06	0.000	0.000	2.23	14.24	9.09	156.79
	65.00	61.43	52.21	52.02	46.27	24	0.0041	0.00	5.17	4.06	0.000	0.000	2.23	14.24	9.37	152.04
	61.43	49.56	52.02	43.65	500	24	0.0167	0.00	5.17	4.06	0.000	0.000	2.23	14.24	18.92	75.30
	49.56	38.81	43.65	33.00	440.72	24	0.0242	0.00	5.17	4.06	0.000	0.000	2.23	14.24	22.73	62.67
	38.81	32.85	33.00	27.29	361.24	24	0.0158	0.00	5.17	4.06	0.000	0.000	2.23	14.24	18.38	77.49
	32.85	31.16	27.29	27.01	275.45	24	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	4.66	305.58
	31.16	32.50	27.01	24.50	209.99	24	0.012	0.00	5.17	4.06	0.000	0.000	2.23	14.24	15.99	89.11
	32.50	36.50	24.50	24.12	381.41	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.37	170.24
	36.50	34.99	24.12	23.74	372.66	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.47	168.27
	34.99	34.00	23.74	23.40	348.28	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.28	171.98

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
	34.00	34.00	23.40	23.05	348.28	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.40	169.50
	34.00	34.00	23.05	22.65	400	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.38	169.92
	34.00	31.50	22.65	22.48	169.62	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.39	169.73
	31.50	29.11	22.48	22.03	446.98	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.41	169.35
	29.11	29.02	22.03	21.76	272.14	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.35	170.59
	29.02	33.15	21.76	21.58	179.72	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.39	169.79
	33.15	32.48	21.58	21.64	338.72	30	-0.0002	0.00	5.17	4.06	0.000	0.000	2.23	14.24	#NUM!	#NUM!
	32.48	32.80	21.64	21.13	114.06	30	0.0045	0.00	5.17	4.06	0.000	0.000	2.23	14.24	17.73	80.36
	32.80	32.00	21.13	21.02	104.85	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.59	165.90
	32.00	32.00	21.02	20.82	203.42	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.31	171.37
	32.00	31.58	20.82	20.54	278.43	30	0.001	0.00	5.17	4.06	0.000	0.000	2.23	14.24	8.41	169.45
HOL	16								5.17	4.06						
	KULSHAN	CREEK	INTERCEPTOR													
	66.00	52.00	63.40	45.41	342	10	0.0526	0.00	0.00	0.00	0.000	0.000	#NUM!	#NUM!	3.25	#NUM!
	52.00	48.00	45.41	40.99	135	10	0.0327	0.00	0.00	0.00	0.000	0.000	#NUM!	#NUM!	2.56	#NUM!
	48.00	46.00	40.99	37.46	107.5	10	0.0328	0.00	0.00	0.00	0.000	0.000	#NUM!	#NUM!	2.57	#NUM!
	46.00	38.00	37.46	30.58	210	10	0.0328	0.00	0.00	0.00	0.000	0.000	#NUM!	#NUM!	2.56	#NUM!
	38.00	37.00	30.58	29.84	72	12	0.0103	0.00	0.00	0.00	0.000	0.000	#NUM!	#NUM!	2.33	#NUM!
N7-A	37.00	36.00	29.84	26.00	350	12	0.011	25.2	0.00	0.00	0.028	0.058	3.34	0.22	2.41	9.18
N7-B	36.00	30.00	26.00	22.32	335	12	0.011	69.40	0.03	0.06	0.076	0.101	3.08	0.59	2.41	24.55
	30.00	34.00	22.32	21.72	130	30	0.0046	0.00	0.10	0.16	0.000	0.000	3.08	0.59	18.01	3.29
	34.00	30.00	21.72	21.35	130	30	0.0028	0.00	0.10	0.16	0.000	0.000	3.08	0.59	14.14	4.19
HOL	17								0.10	0.16						
	14TH	ROW,	RR	TRACKS	TO	ROOSEVELT										
N7-D,E	31.00	24.00	18.03	16.70	330	10	0.004	74.40	0.00	0.00	0.082	0.064	3.32	0.29	0.90	32.71
	24.00	25.00	16.70	15.39	330	10	0.004	0.00	0.08	0.06	0.000	0.000	3.32	0.29	0.89	32.96
HOL	18								0.08	0.06						
	ROOSEVEL	18TH	TO	RR	TRACKS											
REC 17	28.00	28.00	21.35	20.40	190	15	0.005	0.00	0.10	0.16	0.000	0.000	3.08	0.59	2.95	20.07
	28.00	26.00	20.40	18.73	330	15	0.0051	0.00	0.10	0.16	0.000	0.000	3.08	0.59	2.97	19.95
	26.00	25.00	18.73	17.06	334	15	0.005	0.00	0.10	0.16	0.000	0.000	3.08	0.59	2.95	20.07
	25.00	25.00	17.06	15.39	334	15	0.005	0.00	0.10	0.16	0.000	0.000	3.08	0.59	2.95	20.07
REC 18	25.00	27.00	15.39	15.12	270	21	0.001	0.00	0.19	0.22	0.000	0.000	2.99	0.85	3.24	26.30
N8, COMMERCIAL	27.00	25.00	15.12	14.74	380	21	0.001	115.70	0.19	0.22	0.127	0.238	2.80	1.60	3.24	49.50
	25.00	23.00	14.74	14.48	325	21	0.0008	0.00	0.31	0.46	0.000	0.000	2.80	1.60	2.90	55.35
	23.00	26.00	14.48	14.22	325	21	0.0008	0.00	0.31	0.46	0.000	0.000	2.80	1.60	2.90	55.35
N6, COMMERCIAL	26.00	25.00	14.22	13.96	325	21	0.0008	45.7	0.31	0.46	0.050	0.750	2.55	3.45	2.90	119.11
	25.00	27.00	13.96	13.70	325	21	0.0008	0.00	0.36	1.21	0.000	0.000	2.55	3.45	2.90	119.11
HOL	19								0.36	1.21						
	***********	AREA	N5	**												
	URBAN	AVE,	HOAG	TO	ROOSEVELT											
N5-E	30.00	30.00	22.01	19.73	570	8	0.004	54.3	0.00	0.00	0.060	0.059	3.34	0.26	0.49	51.73
N5-D,C	30.00	28.00	19.73	18.83	450	12	0.002	41.40	0.06	0.06	0.046	0.048	3.18	0.44	1.03	43.16
	28.00	28.00	18.83	17.96	435	12	0.002	0.00	0.11	0.11	0.000	0.000	3.18	0.44	1.03	43.16
N5-B	28.00	27.00	17.96	16.81	575	12	0.002	35.6	0.11	0.11	0.039	0.047	3.09	0.62	1.03	60.07
	27.00	27.00	16.81	16.06	375	10	0.002	0.00	0.14	0.15	0.000	0.000	3.09	0.62	0.63	97.69
	27.00	26.00	16.06	15.24	410	15	0.002	0.00	0.14	0.15	0.000	0.000	3.09	0.62	1.87	33.13

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
N5-A,	26.00	26.00	15.24	15.13	50	15	0.0022	43.4	0.14	0.15	0.048	0.057	3.01	0.83	1.96	42.17
	26.00	25.00	15.13	14.49	320	15	0.002	0.00	0.19	0.21	0.000	0.000	3.01	0.83	1.87	44.23
	25.00	26.00	14.49	13.88	305	15	0.002	0.00	0.19	0.21	0.000	0.000	3.01	0.83	1.87	44.23
	26.00	26.00	13.88	13.70	90	15	0.002	0.00	0.19	0.21	0.000	0.000	3.01	0.83	1.87	44.23
HOL	20								0.19	0.21						
	***********	AREA	N4	-	SKAGIT	VALLE	MALL,									
	MARKET	ST.	AND	ROW												
N4-A	28.00	26.00	21.00	19.47	385	8	0.004	8	0.00	0.00	0.009	0.011	3.78	0.05	0.49	9.96
N4-B	27.00	26.00	19.47	18.32	385	10	0.003	45	0.01	0.01	0.050	0.072	3.25	0.33	0.77	42.34
	26.00	26.00	18.32	17.48	380	12	0.0022	0.00	0.06	0.08	0.000	0.000	3.25	0.33	1.08	30.26
	26.00	26.00	17.48	16.75	332	12	0.0022	0.00	0.06	0.08	0.000	0.000	3.25	0.33	1.08	30.35
N4-C	26.00	26.00	16.75	16.16	385	15	0.0015	45	0.06	0.08	0.050	0.060	3.11	0.55	1.63	33.74
	26.00	26.00	16.16	15.84	181	15	0.0018	0.00	0.11	0.14	0.000	0.000	3.11	0.55	1.76	31.41
	26.00	26.00	15.84	15.52	200	15	0.0016	0.00	0.11	0.14	0.000	0.000	3.11	0.55	1.67	33.02
	26.00	26.00	15.52	15.18	212	15	0.0016	0.00	0.11	0.14	0.000	0.000	3.11	0.55	1.67	32.98
	26.00	26.00	15.18	14.84	212	15	0.0016	0.00	0.11	0.14	0.000	0.000	3.11	0.55	1.67	32.98
	26.00	26.00	14.84	14.32	325	15	0.0016	0.00	0.11	0.14	0.000	0.000	3.11	0.55	1.67	33.02
	26.00	26.00	14.32	13.93	244	15	0.0016	0.00	0.11	0.14	0.000	0.000	3.11	0.55	1.67	33.03
N4-D	26.00	26.00	13.93	13.70	144	15	0.0016	27	0.11	0.14	0.030	0.025	3.07	0.65	1.67	39.03
	26.00	26.00	13.70	13.27	269	15	0.0016	0.00	0.14	0.17	0.000	0.000	3.07	0.65	1.67	39.02
	26.00	26.00	13.27	12.71	350	15	0.0016	0.00	0.14	0.17	0.000	0.000	3.07	0.65	1.67	39.00
HOL	21								0.14	0.17						
	***********	AREA	N2,													
	ALDER	LANE	INTERCEPTOR													
REC 19, 20	27.00	24.00	13.70	13.52	300	15	0.0006	0.00	0.56	1.42	0.000	0.000	2.51	4.12	1.02	402.91
	24.00	24.00	13.52	13.34	300	15	0.0006	0.00	0.56	1.42	0.000	0.000	2.51	4.12	1.02	402.91
	24.00	24.00	13.34	13.11	383	30	0.0006	0.00	0.56	1.42	0.000	0.000	2.51	4.12	6.50	63.43
	24.00	24.00	13.11	13.06	20	30	0.0025	0.00	0.56	1.42	0.000	0.000	2.51	4.12	13.26	31.09
	24.00	24.00	13.06	12.91	250	30	0.0006	0.00	0.56	1.42	0.000	0.000	2.51	4.12	6.49	63.45
	24.00	24.00	12.91	12.71	333	30	0.0006	0.00	0.56	1.42	0.000	0.000	2.51	4.12	6.50	63.42
REC 21	24.00	24.00	12.71	12.45	325	30	0.0008	0.00	0.69	1.59	0.000	0.000	2.48	4.63	7.50	61.78
N2,	24.00	24.00	12.45	12.05	500	30	0.0008	25	0.69	1.59	0.028	0.032	2.47	4.73	7.50	63.10
N2,	24.00	24.00	12.05	11.85	250	30	0.0008	10	0.72	1.62	0.011	0.000	2.47	4.74	7.50	63.24
HOL	22								0.73	1.62						
	ALDER	LANE	PUMP	STATION												
	SOUTH	EAST	INTERCEPTOR,	TO	FOWLER											
C2-C	178.15	177.43	169.95	167.23	330	10	0.0082	119	0.00	0.00	0.131	0.072	3.29	0.37	1.29	28.58
	177.35	176.95	167.15	164.55	330	10	0.0079	0.00	0.13	0.07	0.000	0.000	3.29	0.37	1.26	29.23
C2-B	176.60	175.32	164.00	162.68	328.9	18	0.004	65.9	0.13	0.07	0.072	0.029	3.20	0.53	4.30	12.26
	175.32	173.75	162.68	161.34	335.6	18	0.004	0.00	0.20	0.10	0.000	0.000	3.20	0.53	4.29	12.30
HOL	23								0.20	0.10						
	FOWLER	INTERCEPTOR														
	SE	INT.,	FOWLER	TO	BLACKBURN											
S5-D	170.00	170.00	164.60	163.66	314	15	0.003	640	0.00	0.00	0.704	0.198	3.02	1.30	2.28	57.06
	172.00	172.00	163.66	162.83	277	15	0.003	0.00	0.70	0.20	0.000	0.000	3.02	1.30	2.29	57.03
	172.00	172.00	162.83	162.68	52	15	0.0029	0.00	0.70	0.20	0.000	0.000	3.02	1.30	2.24	58.13
	174.00	172.00	162.68	161.92	253	15	0.003	0.00	0.70	0.20	0.000	0.000	3.02	1.30	2.29	56.96

City of Mount Vernon Comprehensive Sewer Plan Update

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
REC 23	173.75	172.50	161.34	159.86	371.1	18	0.004	0.00	0.91	0.30	0.000	0.000	2.91	1.78	4.29	41.54
	172.50	171.65	159.86	158.74	280	18	0.004	0.00	0.91	0.30	0.000	0.000	2.91	1.78	4.29	41.48
	171.65	166.29	158.74	157.37	342.5	18	0.004	0.00	0.91	0.30	0.000	0.000	2.91	1.78	4.29	41.48
	166.29	160.85	157.37	154.65	339.6	18	0.008	0.00	0.91	0.30	0.000	0.000	2.91	1.78	6.08	29.31
HOL	25								0.91	0.30						
	SMALL	PUMP	STATION	ON	BLACKBURN											
	SE	INT.,	BLACKBURN													
REC 25, S5-A,B,C	160.85	145.26	154.60	139.00	398.4	15	0.0392	602.00	0.91	0.30	0.662	0.539	2.65	3.79	8.26	45.86
	145.26	126.70	139.00	120.24	187.6	12	0.1	0.00	1.57	0.84	0.000	0.000	2.65	3.79	7.28	52.03
	126.70	90.00	120.24	82.68	282.4	12	0.133	0.00	1.57	0.84	0.000	0.000	2.65	3.79	8.40	45.11
	90.00	57.81	82.68	52.98	330	12	0.09	0.00	1.57	0.84	0.000	0.000	2.65	3.79	6.91	54.84
	57.81	54.50	52.98	50.70	22.8	12	0.1	0.00	1.57	0.84	0.000	0.000	2.65	3.79	7.28	52.03
	54.50	33.24	50.70	29.64	234	12	0.09	0.00	1.57	0.84	0.000	0.000	2.65	3.79	6.91	54.84
	33.24	31.00	27.66	25.38	22.8	12	0.1	0.00	1.57	0.84	0.000	0.000	2.65	3.79	7.28	52.03
	31.00	21.80	25.38	10.65	147.3	12	0.1	0.00	1.57	0.84	0.000	0.000	2.65	3.79	7.28	52.03
	21.80	17.45	9.15	8.85	299.2	30	0.001	0.00	1.57	0.84	0.000	0.000	2.65	3.79	8.39	45.13
	17.45	16.50	8.85	8.45	396.5	30	0.001	0.00	1.57	0.84	0.000	0.000	2.65	3.79	8.42	44.99
HOL	26								1.57	0.84						
	MT.	VERNON	RD	S.												
	SE	INT.,	BLACKBURN	AT	RR											
REC 26, S4-ABC,S3-ABC,S1-BD	16.50	22.76	8.45	8.27	178	30	0.001	1184.20	1.57	0.84	1.303	1.027	2.44	7.42	8.43	88.01
	22.76	23.00	8.27	8.15	121	30	0.001	0.00	2.87	1.87	0.000	0.000	2.44	7.42	8.35	88.87
	23.00	17.28	8.15	8.02	131	30	0.001	0.00	2.87	1.87	0.000	0.000	2.44	7.42	8.35	88.85
	17.28	16.13	8.02	7.83	188.9	30	0.001	0.00	2.87	1.87	0.000	0.000	2.44	7.42	8.41	88.25
	16.13	14.50	7.83	7.45	378.3	30	0.001	0.00	2.87	1.87	0.000	0.000	2.44	7.42	8.40	88.31
	14.50	13.77	7.45	7.17	275.2	30	0.001	0.00	2.87	1.87	0.000	0.000	2.44	7.42	8.46	87.75
	13.77	14.37	7.17	6.83	279.8	30	0.0012	0.00	2.87	1.87	0.000	0.000	2.44	7.42	9.24	80.29
	14.37	15.36	6.83	6.61	282.6	30	0.0008	0.00	2.87	1.87	0.000	0.000	2.44	7.42	7.40	100.31
	15.36	15.36	6.61	6.60	11	30	0.0009	0.00	2.87	1.87	0.000	0.000	2.44	7.42	7.99	92.83
HOL	27								2.87	1.87						
	BLACKBUF	ROAD														
S2	160.85	149.56	146.25	144.56	348	8	0.0049	104.2	0.00	0.00	0.115	0.061	3.33	0.32	0.54	58.64
	149.56	118.15	144.56	113.63	310	8	0.0998	0.00	0.11	0.06	0.000	0.000	3.33	0.32	2.47	12.94
	118.15	68.21	113.63	62.54	439	8	0.1164	0.00	0.11	0.06	0.000	0.000	3.33	0.32	2.66	11.98
	68.21	53.65	62.54	49.65	142	8	0.0908	0.00	0.11	0.06	0.000	0.000	3.33	0.32	2.35	13.56
	53.65	33.29	49.65	30.34	214	8	0.0902	0.00	0.11	0.06	0.000	0.000	3.33	0.32	2.35	13.60
	33.29	20.37	25.04	11.64	223	8	0.0601	0.00	0.11	0.06	0.000	0.000	3.33	0.32	1.91	16.67
	20.37	17.45	11.64	10.55	276	8	0.0039	0.00	0.11	0.06	0.000	0.000	3.33	0.32	0.49	65.02
S1-C	17.45	16.05	10.55	9.89	272	12	0.0024	26.4	0.11	0.06	0.029	0.015	3.27	0.39	1.13	34.65
	16.05	16.96	9.89	9.60	125	12	0.0023	0.00	0.14	0.08	0.000	0.000	3.27	0.39	1.11	35.44
	16.96	21.34	9.60	9.34	124	12	0.0021	0.00	0.14	0.08	0.000	0.000	3.27	0.39	1.05	37.28
	21.34	17.28	9.34	9.03	305	12	0.001	0.00	0.14	0.08	0.000	0.000	3.27	0.39	0.73	53.54
S1-A,	17.28	16.13	9.03	8.71	190	12	0.0017	48	0.14	0.08	0.053	0.046	3.15	0.58	0.95	61.54
	16.13	15.65	8.71	8.62	108	12	0.0008	0.00	0.20	0.12	0.000	0.000	3.15	0.58	0.66	87.48
	15.65	14.50	8.62	8.02	270	12	0.0022	0.00	0.20	0.12	0.000	0.000	3.15	0.58	1.09	53.57
	14.50	13.77	8.02	7.67	276	15	0.0013	0.00	0.20	0.12	0.000	0.000	3.15	0.58	1.49	39.11
	13.77	14.37	7.67	7.38	180	15	0.0016	0.00	0.20	0.12	0.000	0.000	3.15	0.58	1.68	34.70
	14.37	15.36	7.38	7.21	272	15	0.0006	0.00	0.20	0.12	0.000	0.000	3.15	0.58	1.04	55.71

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
	15.36	15.36	7.21	6.60	11	15	0.0555	0.00	0.20	0.12	0.000	0.000	3.15	0.58	9.83	5.91
SE INT, WALTER - - - - REC 27	15.36	15.30	6.60	6.11	496.9	30	0.001	0.00	3.07	1.99	0.000	0.000	2.42	7.88	8.32	94.67
	15.30	15.50	6.11	5.63	480	30	0.001	0.00	3.07	1.99	0.000	0.000	2.42	7.88	8.38	94.02
	15.50	17.95	5.63	5.13	500	30	0.001	0.00	3.07	1.99	0.000	0.000	2.42	7.88	8.38	94.02
	17.95	17.47	5.13	4.63	497.8	30	0.001	0.00	3.07	1.99	0.000	0.000	2.42	7.88	8.40	93.81
HOL	28	INTERSECT	OF	HAZEL	AND	VALTER			3.07	1.99						
	DOUGLAS															
S1-A,	17.50	17.20	6.60	6.32	220	16	0.0013	84.3	0.00	0.00	0.093	0.052	3.37	0.27	1.77	15.19
	17.20	17.00	6.32	6.12	220	16	0.0009	0.00	0.09	0.05	0.000	0.000	3.37	0.27	1.50	17.98
	17.00	17.50	6.12	6.02	210	16	0.0005	0.00	0.09	0.05	0.000	0.000	3.37	0.27	1.08	24.84
	17.50	17.50	6.02	5.64	320	16	0.0012	0.00	0.09	0.05	0.000	0.000	3.37	0.27	1.71	15.73
	17.50	18.00	5.64	5.56	320	16	0.0003	0.00	0.09	0.05	0.000	0.000	3.37	0.27	0.78	34.28
	18.00	18.50	5.56	5.30	320	16	0.0008	0.00	0.09	0.05	0.000	0.000	3.37	0.27	1.41	19.02
	18.50	19.50	5.30	5.28	260	16	8E-05	0.00	0.09	0.05	0.000	0.000	3.37	0.27	0.43	61.80
	19.50	17.76	5.06	4.26	80	16	0.01	0.00	0.09	0.05	0.000	0.000	3.37	0.27	4.96	5.42
HOL	29	INTERSECT	OF	HAZEL	AND	OUGLAS			0.09	0.05						
	C3-B															
	6TH	ST.	TRUNK,	SOUTH	OF	KINCAID										
C3-B, SOUTH	138.60	124.49	129.91	117.29	300	12	0.0421	29	0.00	0.00	0.032	0.018	3.65	0.10	4.72	2.04
C3-B, NORTH	122.00	85.00	116.67	80.00	300	12	0.1222	38.8	0.03	0.02	0.043	0.034	3.37	0.25	8.05	3.09
	85.00	37.00	80.00	32.00	380	12	0.1263	0.00	0.07	0.05	0.000	0.000	3.37	0.25	8.18	3.04
	37.00	35.00	32.00	30.00	450	21	0.0044	0.00	0.07	0.05	0.000	0.000	3.37	0.25	6.83	3.65
C3-A, EAST	35.00	35.00	30.00	19.00	20	21	0.55	25.5	0.07	0.05	0.028	0.018	3.30	0.33	75.95	0.44
C3-A, WEST	35.00	23.00	19.00	17.85	250	12	0.0046	14.9	0.10	0.07	0.016	0.006	3.27	0.37	1.56	23.39
	23.00	22.00	17.85	14.50	130	12	0.0258	0.00	0.12	0.08	0.000	0.000	3.27	0.37	3.70	9.88
	23.00	21.00	14.50	12.89	850	15	0.0019	0.00	0.12	0.08	0.000	0.000	3.27	0.37	1.82	20.11
	21.00	20.00	12.89	12.34	500	15	0.0011	0.00	0.12	0.08	0.000	0.000	3.27	0.37	1.38	26.39
	20.00	19.00	12.34	10.90	350	18	0.0041	0.00	0.12	0.08	0.000	0.000	3.27	0.37	4.35	8.39
HOL	30								0.12	0.08						
	AREA	N1-A														
N1-A	43.00	41.00	30.00	28.00	125	15	0.016	149.5	0.00	0.00	0.164	0.090	3.23	0.46	5.28	8.64
	41.00	39.00	28.00	26.00	290	15	0.0069	0.00	0.16	0.09	0.000	0.000	3.23	0.46	3.47	13.16
	39.00	37.00	26.00	24.00	350	22	0.0057	0.00	0.16	0.09	0.000	0.000	3.23	0.46	8.76	5.21
	37.00	36.00	24.00	22.00	330	15	0.0061	0.00	0.16	0.09	0.000	0.000	3.23	0.46	3.25	14.04
	32.50	32.00	21.00	19.50	150	15	0.01	0.00	0.16	0.09	0.000	0.000	3.23	0.46	4.18	10.93
HOL	31								0.16	0.09						
	AREA	N1-B														
N1-B	35.00	35.00	24.00	19.50	200	15	0.0225	109.7	0.00	0.00	0.121	0.067	3.30	0.34	6.26	5.46
REC 31	32.00	31.00	19.50	19.00	70	15	0.0071	0.00	0.29	0.16	0.000	0.000	3.08	0.77	3.53	21.83
HOL	32								0.29	0.16						
	DIVISION	ST.	CSO	INTERCEPTOR												
N1-C	25.00	25.00	20.00	19.00	300	20	0.0033	19.7	0.00	0.00	0.022	0.028	3.54	0.12	5.19	2.30
	25.00	25.00	19.00	18.00	220	20	0.0045	0.00	0.02	0.03	0.000	0.000	3.54	0.12	6.06	1.97
	25.00	25.00	16.40	15.40	200	30	0.005	0.00	0.02	0.03	0.000	0.000	3.54	0.12	18.75	0.64
HOL	33	TO CSO REGULATOR							0.02	0.03						

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
	FROM	ALDER	LANE	PUMP	STATION											
	CAMERON	WAY	-	ALDER	LANE	DISCHARGE										
REC 22	27.70	25.42	23.10	22.80	200	27	0.0015	0.00	0.73	1.62	0.000	0.000	2.47	4.74	7.75	61.17
	25.42	28.00	22.80	22.53	180	27	0.0015	0.00	0.73	1.62	0.000	0.000	2.47	4.74	7.75	61.17
	28.00	31.00	22.53	22.02	470	27	0.0011	0.00	0.73	1.62	0.000	0.000	2.47	4.74	6.59	71.92
	31.00	31.00	22.02	21.29	90	27	0.0081	0.00	0.73	1.62	0.000	0.000	2.47	4.74	18.03	26.31
	31.00	33.00	21.29	20.86	180	27	0.0024	0.00	0.73	1.62	0.000	0.000	2.47	4.74	9.78	48.47
HOL	34	<-------	HOLD	FLOWS	FROM	ALDER	LANE		0.73	1.62						
	2ND	ST.														
C1-A,	22.00	22.00	15.50	15.00	280	18	0.0018	15	0.00	0.00	0.017	0.008	3.87	0.05	2.87	1.61
	22.00	22.00	15.00	14.50	280	18	0.0018	0.00	0.02	0.01	0.000	0.000	3.87	0.05	2.87	1.61
	22.00	22.00	14.50	14.00	210	18	0.0024	0.00	0.02	0.01	0.000	0.000	3.87	0.05	3.31	1.39
	22.00	22.00	14.00	13.60	250	18	0.0016	0.00	0.02	0.01	0.000	0.000	3.87	0.05	2.72	1.70
	22.00	22.00	13.60	13.30	180	18	0.0017	0.00	0.02	0.01	0.000	0.000	3.87	0.05	2.77	1.67
	22.00	22.00	13.30	13.00	350	18	0.0009	0.00	0.02	0.01	0.000	0.000	3.87	0.05	1.99	2.32
	22.00	22.00	13.00	12.50	330	18	0.0015	0.00	0.02	0.01	0.000	0.000	3.87	0.05	2.64	1.75
	22.00	22.00	12.50	12.00	330	18	0.0015	0.00	0.02	0.01	0.000	0.000	3.87	0.05	2.64	1.75
	22.00	22.00	11.50	11.00	330	18	0.0015	0.00	0.02	0.01	0.000	0.000	3.87	0.05	2.64	1.75
	22.00	22.00	11.00	10.00	260	18	0.0038	0.00	0.02	0.01	0.000	0.000	3.87	0.05	4.21	1.10
HOL	35	TO CSO REGULATOR							0.02	0.01						
	AREA	N3	-	FREEWAY	DRIVE											
	NORTH	1ST	STREET,	FREEWAY	TO	DIVISION										
	DOWNTOW	1ST	STREET,	DIVISION	TO	KINCAID										
	CLEVELAN	KINCAID	TO	PARK	ST.											
	CLEVELAN	PARK	TO	HAZEL												
N3	34.00	34.00	26.97	26.07	321	8	0.0028	151.2	0.00	0.00	0.166	0.213	3.00	0.81	0.41	195.01
	34.00	34.00	26.07	25.14	332	12	0.0028	0.00	0.17	0.21	0.000	0.000	3.00	0.81	1.22	66.17
	34.00	34.00	25.14	24.22	329	10	0.0028	0.00	0.17	0.21	0.000	0.000	3.00	0.81	0.75	107.70
	34.00	34.00	24.22	23.30	329	10	0.0028	0.00	0.17	0.21	0.000	0.000	3.00	0.81	0.75	107.70
	34.00	34.00	23.30	22.30	330	10	0.003	0.00	0.17	0.21	0.000	0.000	3.00	0.81	0.78	103.46
REC 34	33.00	30.80	20.86	20.48	250	27	0.0015	0.00	0.90	1.83	0.000	0.000	2.44	5.38	7.80	68.90
	30.80	33.20	20.48	20.03	300	27	0.0015	0.00	0.90	1.83	0.000	0.000	2.44	5.38	7.75	69.36
	32.20	32.00	20.03	19.32	280	27	0.0025	0.00	0.90	1.83	0.000	0.000	2.44	5.38	10.08	53.34
	32.00	31.00	19.32	19.07	420	27	0.0006	0.00	0.90	1.83	0.000	0.000	2.44	5.38	4.88	110.10
	31.00	28.00	19.07	18.51	380	27	0.0015	0.00	0.90	1.83	0.000	0.000	2.44	5.38	7.68	69.97
	28.00	28.50	18.51	18.03	300	27	0.0016	0.00	0.90	1.83	0.000	0.000	2.44	5.38	8.01	67.16
	28.50	28.50	18.03	17.93	50	27	0.002	0.00	0.90	1.83	0.000	0.000	2.44	5.38	8.95	60.07
	28.00	28.00	17.62	17.00	170	24	0.0036	0.00	0.90	1.83	0.000	0.000	2.44	5.38	8.83	60.89
	25.50	24.50	17.68	17.13	186	24	0.003	0.00	0.90	1.83	0.000	0.000	2.44	5.38	7.95	67.63
	24.50	24.00	17.13	16.37	238	24	0.0032	0.00	0.90	1.83	0.000	0.000	2.44	5.38	8.26	65.08
	24.00	24.00	16.37	15.52	265	24	0.0032	0.00	0.90	1.83	0.000	0.000	2.44	5.38	8.28	64.93
	24.00	24.20	15.52	14.73	245.9	24	0.0032	0.00	0.90	1.83	0.000	0.000	2.44	5.38	8.29	64.88
	24.20	23.50	14.73	13.93	244	24	0.0033	0.00	0.90	1.83	0.000	0.000	2.44	5.38	8.37	64.22
	23.50	22.20	13.93	13.13	248	24	0.0032	0.00	0.90	1.83	0.000	0.000	2.44	5.38	8.30	64.75
C1-A	22.20	21.10	12.63	12.00	330	30	0.0019	71	0.90	1.83	0.078	0.118	2.43	5.71	11.58	49.31
	21.10	20.60	12.00	11.36	331	30	0.0019	0.00	0.98	1.95	0.000	0.000	2.43	5.71	11.66	48.99
	20.60	20.00	11.36	10.68	247	30	0.0028	0.00	0.98	1.95	0.000	0.000	2.43	5.71	13.91	41.06
	20.00	18.10	10.68	10.04	325	30	0.002	0.00	0.98	1.95	0.000	0.000	2.43	5.71	11.76	48.55
	18.10	17.50	10.04	9.12	297	27	0.0031	0.00	0.98	1.95	0.000	0.000	2.43	5.71	11.14	51.26

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
	20.00	20.00	11.60	11.50	20	27	0.005	0.00	0.98	1.95	0.000	0.000	2.43	5.71	14.15	40.35
	19.50	19.50	9.12	8.05	340	24	0.0031	0.00	0.98	1.95	0.000	0.000	2.43	5.71	8.20	69.63
HOL	36								0.98	1.95						
	KINCAID STREET, I5 TO BNRR															
REC 30	27.80	23.40	22.60	15.99	104	30	0.0636	0.00	0.12	0.08	0.000	0.000	3.27	0.37	66.83	0.55
	23.40	22.50	15.99	13.63	530	30	0.0045	0.00	0.12	0.08	0.000	0.000	3.27	0.37	17.69	2.07
	22.50	22.00	13.63	13.20	40	30	0.0108	0.00	0.12	0.08	0.000	0.000	3.27	0.37	27.49	1.33
HOL	37															
	CSO REGULATOR - CAMERON WAY TO HAZEL															
REC 16, 33, 35	30.19	31.29	20.41	19.92	322.85	60	0.0015	0.00	5.21	4.10	0.000	0.000	2.23	14.35	65.58	21.89
	31.29	31.66	19.92	19.41	333.28	60	0.0015	0.00	5.21	4.10	0.000	0.000	2.23	14.35	65.85	21.80
	31.66	32.00	19.41	18.96	297.58	60	0.0015	0.00	5.21	4.10	0.000	0.000	2.23	14.35	65.46	21.93
	32.00	31.21	18.96	18.69	177.42	60	0.0015	0.00	5.21	4.10	0.000	0.000	2.23	14.35	65.67	21.86
	31.21	31.00	18.69	18.67	12	60	0.0017	0.00	5.21	4.10	0.000	0.000	2.23	14.35	68.72	20.89
	31.00	31.00	18.67	18.12	367.21	60	0.0015	0.00	5.21	4.10	0.000	0.000	2.23	14.35	65.15	22.03
	31.00	31.00	18.12	18.07	36.08	60	0.0014	0.00	5.21	4.10	0.000	0.000	2.23	14.35	62.66	22.90
	31.00	28.90	18.07	17.98	55.33	60	0.0016	0.00	5.21	4.10	0.000	0.000	2.23	14.35	67.89	21.14
	28.90	31.00	17.98	17.36	416.02	60	0.0015	0.00	5.21	4.10	0.000	0.000	2.23	14.35	64.98	22.09
	31.00	26.00	17.36	17.18	29.49	60	0.0061	0.00	5.21	4.10	0.000	0.000	2.23	14.35	131.51	10.91
	26.00	26.00	17.18	17.10	8	60	0.01	0.00	5.21	4.10	0.000	0.000	2.23	14.35	168.33	8.53
	26.00	26.00	17.10	16.54	187.47	60	0.003	0.00	5.21	4.10	0.000	0.000	2.23	14.35	92.00	15.60
	26.00	26.00	16.54	16.35	62.8	60	0.003	0.00	5.21	4.10	0.000	0.000	2.23	14.35	92.59	15.50
	26.00	26.00	16.35	16.31	8	60	0.005	0.00	5.21	4.10	0.000	0.000	2.23	14.35	119.03	12.06
	26.00	26.00	16.31	15.77	121.18	60	0.0045	0.00	5.21	4.10	0.000	0.000	2.23	14.35	112.37	12.77
	26.00	26.00	15.77	15.32	100	60	0.0045	0.00	5.21	4.10	0.000	0.000	2.23	14.35	112.92	12.71
	26.00	26.00	15.32	15.01	87.69	60	0.0035	0.00	5.21	4.10	0.000	0.000	2.23	14.35	100.08	14.34
	26.00	26.00	15.01	14.82	54.24	60	0.0035	0.00	5.21	4.10	0.000	0.000	2.23	14.35	99.63	14.41
	26.00	24.50	14.82	14.08	458.07	60	0.0016	0.00	5.21	4.10	0.000	0.000	2.23	14.35	67.66	21.21
REC 37	24.50	24.00	14.08	12.10	475	60	0.0042	0.00	5.21	4.10	0.000	0.000	2.23	14.35	108.68	13.21
	24.00	24.00	12.10	11.06	388.06	60	0.0027	0.00	5.21	4.10	0.000	0.000	2.23	14.35	87.14	16.47
	24.00	24.00	11.06	9.95	303.31	60	0.0037	0.00	5.21	4.10	0.000	0.000	2.23	14.35	101.83	14.09
	24.00	24.00	9.95	8.80	533.69	60	0.0022	0.00	5.21	4.10	0.000	0.000	2.23	14.35	78.14	18.37
	24.00	24.00	8.80	7.91	421.82	60	0.0021	0.00	5.21	4.10	0.000	0.000	2.23	14.35	77.32	18.56
	24.00	24.00	7.91	7.67	53.29	60	0.0045	0.00	5.21	4.10	0.000	0.000	2.23	14.35	112.96	12.71
	24.00	24.00	7.67	7.62	8	60	0.0062	0.00	5.21	4.10	0.000	0.000	2.23	14.35	133.08	10.79
	24.00	24.00	7.62	7.08	207.36	60	0.0026	0.00	5.21	4.10	0.000	0.000	2.23	14.35	85.90	16.71
	24.00	18.90	7.08	6.56	347.53	60	0.0015	0.00	5.21	4.10	0.000	0.000	2.23	14.35	65.11	22.04
	18.90	18.00	6.56	6.51	37	60	0.0014	0.00	5.21	4.10	0.000	0.000	2.23	14.35	61.88	23.19
	18.00	17.60	6.51	6.08	286	60	0.0015	0.00	5.21	4.10	0.000	0.000	2.23	14.35	65.27	21.99
REC 36	17.60	17.70	6.08	5.67	233	60	0.0018	0.00	6.18	6.05	0.000	0.000	2.13	19.07	70.61	27.01
C1-C,	17.70	17.70	5.67	5.21	312.86	60	0.0015	22	6.18	6.05	0.024	0.051	2.13	19.19	64.54	29.73
	17.70	17.40	5.21	5.14	33.3	42	0.0021	0.00	6.21	6.10	0.000	0.000	2.13	19.19	29.81	64.37
	17.40	17.70	5.14	4.70	294.76	42	0.0015	0.00	6.21	6.10	0.000	0.000	2.13	19.19	25.12	76.39
	17.70	17.20	4.70	4.30	270	42	0.0015	0.00	6.21	6.10	0.000	0.000	2.13	19.19	25.03	76.68
HOL	38								6.21	6.10						
	I-5	XING	AT	WELLS	NURSEI	PARK										
	HAZEL,	CLEVELAND	TO	WWTP												
C2-A	18.00	18.00	14.00	13.00	310	18	0.0032	70.4	0.00	0.00	0.077	0.027	3.54	0.17	3.86	4.48

Drainage Segment	1 Upstream MH GRD	2 Down-stream MH GRD	3 Up-stream MH IE	4 Down-stream MH IE	5 Length	6 Dia-meter	7 Slope	13 Service Area (ac)	14 Upstream Infilt (mgd)	15 Upstream Avg San (mgd)	16 Infilt (mgd)	17 Avg San Flow (mgd)	18 Peak Factor	19 Peak Flow (mgd)	20 Avail Cap (mgd)	21 Percent Utilized
C1-D	18.00	18.90	13.00	12.20	280	18	0.0029	30.1	0.08	0.03	0.033	0.015	3.43	0.25	3.63	6.98
C1-B	18.90	19.00	12.20	12.10	470	21	0.0002	28.9	0.11	0.04	0.032	0.053	3.21	0.45	1.49	29.93
	19.00	19.50	12.10	12.00	400	21	0.0002	0.00	0.14	0.09	0.000	0.000	3.21	0.45	1.62	27.61
	19.50	19.50	12.00	11.90	150	21	0.0007	0.00	0.14	0.09	0.000	0.000	3.21	0.45	2.64	16.91
	19.50	19.00	11.90	11.80	140	21	0.0007	0.00	0.14	0.09	0.000	0.000	3.21	0.45	2.74	16.33
	19.00	19.00	11.80	11.70	140	21	0.0007	0.00	0.14	0.09	0.000	0.000	3.21	0.45	2.74	16.33
	19.00	18.00	11.70	11.60	140	21	0.0007	0.00	0.14	0.09	0.000	0.000	3.21	0.45	2.74	16.33
	17.53	17.40	8.83	7.20	357.98	30	0.0046	0.00	0.14	0.09	0.000	0.000	3.21	0.45	17.89	2.50
	17.40	17.40	7.20	5.97	269.62	30	0.0046	0.00	0.14	0.09	0.000	0.000	3.21	0.45	17.91	2.50
REC 28, 38	17.47	17.78	4.32	4.19	132.8	42	0.001	0.00	9.42	8.18	0.000	0.000	2.05	26.21	20.35	128.83
	17.78	17.80	4.19	4.06	101.7	42	0.0013	0.00	9.42	8.18	0.000	0.000	2.05	26.21	23.25	112.74
	17.80	17.80	4.06	4.03	34.8	42	0.0009	0.00	9.42	8.18	0.000	0.000	2.05	26.21	19.09	137.29
REC 29	17.80	19.50	4.03	3.66	300	42	0.0012	0.00	9.51	8.23	0.000	0.000	2.05	26.40	22.84	115.59
	19.50	19.50	3.46	3.40	40	42	0.0015	0.00	9.51	8.23	0.000	0.000	2.05	26.40	25.18	104.82
	20.00	20.00	3.30	2.97	224	42	0.0015	0.00	9.51	8.23	0.000	0.000	2.05	26.40	24.96	105.76
	20.00	20.00	2.87	2.60	178	42	0.0015	0.00	9.51	8.23	0.000	0.000	2.05	26.40	25.33	104.23
	20.00	20.00	2.50	2.00	39	42	0.0128	0.00	9.51	8.23	0.000	0.000	2.05	26.40	73.63	35.85
END									9.51	8.23						
WEST MOUNT VERNON PUMP STATION																
WEST MV-W1-A, W2	18	18	14	13	310	18	0.0032	528.00	0.00	0.00	0.581	0.429	2.82	1.79	3.86	46.47
END																

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

JAN 02 2000

DALE K. ROUNDY

ATTORNEYS AT LAW

Business Planning
Real Estate/Land Use
Employment Relations
Commercial Transactions
Construction
Estate Planning
Probate
Municipal
Civil Litigation
Condemnation

506 NORTH MAIN
P.O. BOX 1500
COUPEVILLE, WASHINGTON 98239
Telephone: 360•678•6200
Facsimile: 360•678•2298

E-mail: roundy @whidbey.net

Dale K. Roundy

Michael L. Charneski
Of Counsel

Nanette E. Streubel
Paralegal

December 20, 2000

RECEIVED
CITY OF MT. VERNON

DEC 28 2000

WASTE WATER TREATMENT

Walt Enquist
City of Mount Vernon
Waste Water Utility
Post Office Box 809
Mount Vernon, Washington 98273

Re: **Draper Valley Farms, Inc.**
Draft Industrial Pretreatment Report

Dear Walt:

Draper Valley Farms has received and reviewed the draft Industrial Pretreatment Report prepared by HDR Engineering dated October, 2000 and has asked that I forward to you the enclosed copy of the draft containing Draper Valley Farms' comments.

On an overall basis, the Company feels that the draft report provides a fair and thorough review of the Company's pretreatment system. In addition to the Company's comments are shown on the draft, the Company has asked me to point out that the "potential improvement" listed at Section VIII(A)(2) has been evaluated and it has been determined that automating the flow would be extremely expensive and not necessarily more effective than manual shut off of the water flow. The Company has also considered the recommendation found at VIII(B)(1) concerning the submersible pumps and has determined that regular monitoring of the systems by Company personnel will allow for proper maintenance and control of the lift station and the proper feeding of chemicals. Neither of these comments require a modification to the draft report, but the Company did want you to be aware of the action they have taken in these two specific areas.

Draper Valley Farms looks forward to receiving the final report. Thank you for your efforts in arranging to have HDR Engineering provide its

Re: **Draper Valley Farms, Inc.**
Draft Industrial Pretreatment Report

assessment and recommendations concerning the sewer treatment system and the Company's pretreatment facilities. Best wishes for the holidays!

Sincerely,

Dale K. Roundy

DKR:nes
Enclosure
cc: John Jefferson, Draper Valley Farms, Inc.

Appendix F

Meeting Minutes from January 9, 2001, Meeting Between City of Mount Vernon Staff, Department of Ecology Representatives, and HDR Engineering

City of Mount Vernon
Department of Ecology Meeting
Draft

Date:	**January 9, 2001**
Time:	**10:00 am to 1:00 pm**
Location:	**Engineering Conference Room**
Purpose of Meeting:	**Discussion of Wastewater Permitting & Planning Issues**

Attendance:

Jerry Shervey & Bernard Jones, Department of Ecology
Brad Einfeld & Jason Sharpley, HDR Engineering
Bill Fox, Cosmopolitan Engineering
Walt Enquist, City of Mount Vernon, Wastewater
Bill Fullner, City of Mount Vernon, Wastewater
John Buckley, City of Mount Vernon Public Works
Michele Garcia, City of Mount Vernon, Admin. Assistant

Discussion Items:

1. **Revised Mixing Zone Model Results:** Bill Fox distributed a report on his modeling results. His results differed from Jerry Shervey's because of a CFS to MGD conversion over site by Jerry Shervey and discrepancies in effluent metals data. Jerry said he would check his figures and rerun the model. Bill Fullner said he would confirm the metals data and find out where differing numbers originated. Jerry said that there would definitely be limits for several metals, including copper, which would be difficult to meet. He suggested that Mount Vernon research ultra clean sampling and testing methods.

   Mount Vernon will need to request a compliance schedule to meet the NPDES limits proposed. This can be done during the NPDES draft review process. Jerry may include a compliance schedule with the draft permit.

2. **Draft Sewer Comprehensive Plan – Flow Projections:** Brad Einfeld reviewed the allowance for present I&I for the combined system. Brad distributed a Flow Analysis showing the flow components and calculations used to determine the I&I component. Much discussion was held regarding summer and winter flows and infiltration and inflow allowances. Brad explained that the City has a combined sewer/storm system and this creates a high I & I component. Jerry Shervey was concerned that the 5.39 mgd from January 1997 was not an accurate representation of the current Average Day Maximum Month (ADMM) flow. Jerry was concerned that the flow was skewed during the initial operation of the CSO regulator. Walt pointed out that in 1999 the rainfall was near average and the CSO regulator was being operated in accordance with design assumptions with a ADMM flow of 5.11 mgd. In 2000 the rainfall was 30% below average with an ADMMW flow of 4.52 mgd. It was agreed that 5.39 is a reasonable number for current ADMM flow conditions.

Jerry agreed with the flow projections Brad presented in the Flow Analysis.

3. **Wastewater Treatment Plant Flow Re-Rating:** SEPA is just about finished. Walt Enquist stated that he would add river improvement information.

4. **NPDES Requirements for 85% BOD & TSS Reduction During Times of Dilute Influent:** Exemption in affect only from November through June.

5. **The CSO Baseline Flow referenced in the Consent Order is based on the Comprehensive Sewer Plan (Will this be Mount Vernon's baseline through January 1, 2015?):** Mount Vernon's baseline will not change from what was has been established in the 1995 Comprehensive Sewer and Combined Sewer Overflow reduction plan, figure V-16. This will be included in the NPDES Fact Sheet.

6. **Will the NPDES permit include an exchange ratio for Ammonia/CBOD:** This can't be included in the permit, but can be revised during the NPDES renewal process.

7. **BOD or CBOD – Will Mount Vernon have the option of selecting one or the other:** Yes. If the City chooses to use CBOD then they will have to submit a formal request to do so. Jerry will need to know which option the City chooses for the next permit cycle.

8. **Does DOE have discretion in setting the effluent pH limit? Is pH 6.0 mandatory? pH adjustment will be necessary during periods of nitrification:** The DOE does not have discretion in changing the effluent pH limit. pH limit of 6.0 may get higher.

9. **Will TMDL limits apply during occasions when storm events occur and the river flow remains below 6000 cfs i.e. September or October storm when overflows could occur and river is below 6000 cfs):** Data will have to be reviewed to determine total daily load for historical information (plant + CSO's) under these conditions. Daily limits will apply when river flows are below 6000 cfs. Monthly limits will apply during TMDL periods (July through October), regardless of flow.

10. **Funding outlook for CSO improvements, and anti-degradation improvements:** Bernard stated that grant funding is unlikely, however, loan options are a possibility. Review time for facility plans could be from 30 – 60 days.

Facility plan is about 95% complete and should be ready in 1 – 3 weeks.

John Buckley inquired as to the status of where the outfall design proposal is currently. Brad Einfeld stated that it will be submitted to the City the week of January 16. The preliminary design will address, issues i.e., as to whether to use one pipe or two, etc.

DNR Easement is supposed to expire on July 31, 2001. Bill Fox will provide support for DNR easement permit during the application process.

If there are any inaccuracies or misunderstandings, please contact Walt Enquist at (360) 336-6219.

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

STATE OF WASHINGTON
DEPARTMENT OF ECOLOGY
Northwest Regional Office • 3190 160th Avenue SE • Bellevue, Washington 98008-5452 • (425) 649-7000

September 4, 2001

CERTIFIED MAIL
7001 0320 0000 4654 0972

The Honorable Skye Richendrfer
Mayor, City of Mt. Vernon
PO Box 809
Mt. Vernon, WA 98273

Dear Mayor Richendrfer:

RE: NPDES Permit Issuance
City of Mount Vernon Wastewater Treatment Plant; Permit No. WA-002407-4
Expiration Date: June 30, 2003

Under the provisions of Chapter 90.48 RCW Water Pollution Control Laws as amended and the Federal Water Pollution Control Act (The Clean Water Act) Title 33 United States Code, Section 1251 et seq., the enclosed NPDES Permit No. WA-002407-4 is hereby issued to the City of Mount Vernon Wastewater Treatment Plant located at 1401 Britt Road, Mount Vernon, Washington (Skagit County).

The permit authorizes the Permittee to discharge secondary treated and disinfected effluent into the Skagit River subject to the terms and conditions of the permit.

Pursuant to RCW 90.48.465, a permit fee will be assessed. Semi-annual notices for payment will be mailed to you from our office in Olympia.

Any person feeling aggrieved by this NPDES permit may obtain review thereof by application, within 30 days of receipt of this permit, to the Washington Pollution Control Hearings Board, Post Office Box 40903, Olympia, WA 98504-0903. Concurrently, a copy of the application must be sent to the Department of Ecology, Post Office Box 47600, Olympia, WA 98504-7600. These procedures are consistent with the provisions of Chapter 43.21B RCW and the rules and regulations adopted thereunder.

The Honorable Skye Richendrfer
Mayor, City of Mt. Vernon
September 4, 2001
Page 2

Any appeal must contain the following in accordance with the rules of the hearings board:

a) The appellant's name and address;
b) The date and number of the permit appealed;
c) A description of the substance of the permit, that is the subject of the appeal;
d) A clear, separate, and concise statement of every error alleged to have been committed;
e) A clear and concise statement of facts which the requester relies to sustain his or her statements of error;
f) A statement setting forth the relief sought; and
g) A copy of the order, decision, or application appealed from.

An application for permit renewal must be made at least 180 days prior to the expiration date of this permit. Copies of the Discharge Monitoring Report (DMR) forms have been forwarded to Walt Enquist, Wastewater Superintendent along with a copy of the permit. If at any time during the term of this permit a question should arise regarding the permit or discharge, or if there is a significant change in the discharge or operation, please contact Bernard Jones at (425) 649-7146.

Sincerely,

Kevin C. Fitzpatrick

Kevin C. Fitzpatrick
Water Quality Section Manager
Northwest Regional Office

KCF:tm
Enclosures

cc: Mr. Walt Enquist, Wastewater Utility Supervisor
Bev Poston, Permit Fee Unit
Laura Fricke, Municipal Unit Supervisor
Bernard Jones, Facility Manager
Chris Smith, WPLCS
Central Files: WQ 1.1, WA-002407-4

Issuance Date: September 4, 2001
Effective Date: October 1, 2001
Expiration Date: June 30, 2003

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT No. WA-002407-4

State of Washington
DEPARTMENT OF ECOLOGY
Northwest Regional Office
3190 – 160th Avenue SE
Bellevue, WA 98008-5452

In compliance with the provisions of
The State of Washington Water Pollution Control Law
Chapter 90.48 Revised Code of Washington
and
The Federal Water Pollution Control Act
(The Clean Water Act)
Title 33 United States Code, Section 1251 et seq.

City of Mount Vernon
P. O. Box 809
Mount Vernon, Washington 98273

Plant Location: 1401 Britt Road Mount Vernon, WA	Receiving Water: Skagit River
Water Body I.D. No.: WA-03-1010	Discharge Location – Outfall #1: Latitude: 48° 24' 48" N Longitude: 122° 20' 06" W
Plant Type: Publicly owned municipal wastewater treatment plant -- conventional mixed activated sludge	

is authorized to discharge in accordance with the special and general conditions that follow.

Kevin C Fitzpatrick

Kevin C. Fitzpatrick
Water Quality Section Manager
Northwest Regional Office
Washington State Department of Ecology

TABLE OF CONTENTS

GENERAL CONDITIONS

SUMMARY OF PERMIT REPORT SUBMITTALS

Refer to the Special and General Conditions of this permit for additional submittal requirements.

Permit Section	Submittal	Frequency	First Submittal Date
S2.A	Discharge Monitoring Report	Monthly	November 15, 2001
S2.A	Discharge Summary Report	Monthly	November 15, 2001
S2.B	Priority Pollutant Metals	Quarterly, begin with 4^{th} quarter of 2001	February 15, 2002
S2.F	Metals Sampling and Analysis Report	One time only	By January 1, 2002 (sample by 12-30-01)
S3.G	Compliance Progress Report	Annually	April 1, 2002
S6.F	Industrial User Survey – submit with application for permit renewal	1/permit cycle	December 30, 2002
S8.A, S8.F	Acute Toxicity Characterization Reports	Quarterly for one year, begin with 4^{th} quarter of 2001	December 30, 2001 (sample by 10-30-01)
S8.C, S8.F	Acute Toxicity Compliance Monitoring Reports	Quarterly, if required	Depends on test results
S9.A, S9.F	Chronic Toxicity Characterization Data	Twice per year, begin with 2^{nd} half of 2001	December 30, 2001 (sample by 10-30-01)
S9.C, S9.F	Chronic Toxicity Compliance Monitoring Reports	Twice per year, if required	Depends on test results
S10.B	CSO Sampling Plan update	1/permit cycle	November 1, 2001
S10.B	Annual CSO Report	Annually	April 1, 2002
S10.C	CSO Plan amendment	1/permit cycle	December 30, 2002
G7.	Application for Permit Renewal	1/permit cycle	December 30, 2002

Other submittals may be required as a consequence of treatment system failure or bypass, construction or maintenance activities, additional loading to the plant, or other conditions contained in this permit.

SPECIAL CONDITIONS

S1. DISCHARGE LIMITATIONS

A. Effluent Limitations

All discharges and activities authorized by this permit shall be consistent with the terms and conditions of this permit. The discharge of any of the following pollutants at a concentration or mass in excess of that authorized by this permit shall constitute a violation of the terms and conditions of this permit.

Beginning on the effective date of this permit and lasting through the expiration date, the Permittee is authorized to discharge municipal wastewater from Outfall #1 subject to the limitations listed in Tables S1.1 and S1.2 of this condition:

The limits for pH and copper are subject to a compliance schedule – the Permittee shall report annually to the Department on progress towards meeting the final effluent limits (see S3.G). Interim limits as shown below shall be in effect during the term of this permit. The Permittee shall report to the Department by June 30, 2003, how the final limits for pH and copper will be met. The report may be incorporated in a facility plan, engineering report, or submitted as a separate document. See S8 and S9 for standards and requirements for whole effluent toxicity testing.

Table S1.1: Basic Effluent Limitations[a] at OUTFALL #1

Parameter	Average Monthly	Average Weekly
5-day Biochemical Oxygen Demand[b] (BOD_5)	30 mg/L, 1401 lbs./day	45 mg/L, 2102 lbs./day
Total Suspended Solids[b] (TSS)	30 mg/L, 1401 lbs./day	45 mg/L, 2102 lbs./day
Fecal Coliform Bacteria	200 cfu /100 mL	400 cfu /100 mL
pH (interim)	shall not be outside the range 6.0 to 9.0	
pH[c]	***shall not be outside the range 6.6 to 9.0***	
Parameter	**Average Monthly**	**Maximum Daily**
NH_3-N (as N)	31 mg/L, 1448 lbs./day	41 mg/L
Copper (interim limit)	21.3 ug/L, 1 lbs./day	35 ug/L
copper[c]	*9.4 ug/L, 0.44 lbs./day*	*16.6 ug/L*
Zinc	88.4 ug/L, 4.13 lbs./day	177.4 ug/L
Total Residual Chlorine	0.05 mg/L, 2.21 lbs./day	0.1 mg/L

[a]The average monthly and weekly effluent limitations are based on the arithmetic mean of the samples taken with the exception of fecal coliform, which is based on the geometric mean.

[b]The average monthly effluent concentration for BOD_5 and TSS shall not exceed 30 mg/L or 20 percent of the respective monthly average influent concentrations, whichever is more stringent.

[c]Final limit. The interim limit shall be in effect during the term of this permit.

The average monthly limitations in Table S1.2 shall be in effect for the months of July, August, September, and October. The maximum daily limitations in Table S1.2 shall be in effect from July 1 to November 15 on any day that the average flow for the day of the Skagit River is below 6000 cubic feet per second. Skagit River flow is the flow measured at US Geological Survey (USGS) gauging station number 12200500 named "Skagit River near Mount Vernon" and listed by USGS as located at Latitude 48 ° 26' 42" and Longitude 122° 20' 03". The maximum daily limitations shall be in effect if the flow data is unavailable for this station.

Table S1.2: Low River Flow Effluent Limitations[a] at OUTFALL #1 from July 1 to November 15.

Parameter	Average Monthly[a]	Maximum Daily[b]
Ammonia as Nitrogen (N)	922 lbs./day	1188 lbs./day
[a]The average monthly limitations are based on the arithmetic mean of the samples taken. The average monthly ammonia limit listed here is in effect for the months of July, August, September, and October. [b]The maximum daily limit applies on any day that the average flow for the day of the Skagit River is below 6000 cubic feet per second.		

B. Mixing Zone Descriptions

The boundaries of the mixing zone are limited to 307 feet downstream of the outfall diffuser. The estimated dilution factor is 35 to 1.

The zone of acute criteria exceedance is limited to 31 feet downstream of the outfall diffuser. The estimated dilution factor is 5 to 1.

S2. MONITORING REQUIREMENTS

The Permittee shall monitor the wastewater according to the schedule in Table S2A.1 throughout the year and increase the monitoring frequency as shown in Table S2A.2 from July 1 through November 15. Sampling frequencies listed as quarterly, twice per year, and once per year shall be based on the calendar year (January through March, April through June, etc.)

A. Compliance Monitoring

Table S2A.1: Base Monitoring Requirements – applicable throughout the year

Tests	Sample Point	Minimum Sampling Frequency	Sample Type
Flow (MGD)	influent effluent	continuous continuous	on-line on-line
BOD_5	influent effluent	3/week 3/week	24-hour composite 24-hour composite
TSS	influent final effluent	3/week 3/week	24-hour composite 24-hour composite
Fecal Coliform Bacteria	final effluent	5/week	Grab
pH	final effluent	daily	Grab
Ammonia	final effluent	1/week	24-hour composite
Total Residual Chlorine	final effluent	5/week	Grab
Copper	final effluent	2/month, at least one (1) week apart	24-hour composite
Zinc	final effluent	2/month, at least one (1) week apart	24-hour composite
Acute whole effluent toxicity[1]	final effluent	1/3 months	24-hour composite
Chronic whole effluent toxicity[2]	final effluent	1/6 months	24-hour composite

[1]See permit section S8 for requirements for acute whole effluent toxicity testing.

[2]See permit section S9 for requirements for chronic whole effluent toxicity testing.

Table S2A.2: Additional Low River Flow Monitoring Requirements for summer season (applicable from July 1 through November 15)

Tests	Sample Point	Minimum Sampling Frequency	Sample Type
Ammonia as N	effluent	3/week	24-hour composite

Report concentration and mass discharge for ammonia. Effluent ammonia and BOD_5 samples shall be taken during the same 24-hour period and from the same location. Note the requirement for checking and reporting daily receiving water flows in S3.B

B. Characterization Monitoring

The Permittee shall perform the following effluent characterizations. These samples shall be taken together at one time; one of the quarterly metals samples shall be collected at the same time as the other samples required here. The grab sample for volatile organics shall be collected during the time the 24-hour composites are collected. The copper and zinc data to satisfy this requirement may be used to meet the sampling requirements in S2A1.

Tests	Sample Location	Sampling Frequency	Sample Type
Priority Pollutant Metals [1]	final effluent	1 per quarter	24-hour composite
Mercury[2]	final effluent	1 per quarter	24-hour composite
Organics: Acid extractable Base-Neutral Pesticides Polychlorinated-Biphenyles	final effluent	1 per year	24-hour composite
Volatile Organics	final effluent	1 per year	grab

[1]Priority pollutant metals shall include: Antimony, Arsenic, Beryllium, Cadmium, Chromium, Copper, Lead, Nickel, Selenium, Silver, Thallium, and Zinc. Analysis shall be by Standard Method number 200.7 - Inductively Coupled Plasma (ICP) or other method that provides lower detection levels than ICP.

[2] Mercury analysis shall be by Standard Method number 245.1 or 245.2 using cold vapor extraction. The method detection level (MDL) for mercury is 0.2 ug/L using cold vapor extraction absorption spectrometry and method number 245.1 or 245.2 from 40 CFR 136. The quantitation level (QL) for mercury is 1 ug/L (5 x MDL).

C. Sampling and Analytical Procedures

Samples and measurements taken to meet the requirements of this permit shall be representative of the volume and nature of the monitored parameters, including representative sampling of any unusual discharge or discharge condition, including bypasses, upsets, and maintenance-related conditions affecting effluent quality.

Sampling and analytical methods used to meet the water and wastewater monitoring requirements specified in this permit shall conform to the latest revision of the *Guidelines Establishing Test Procedures for the Analysis of Pollutants* contained in 40 CFR Part 136 or to the latest revision of *Standard Methods for the Examination of Water and Wastewater* (APHA), unless otherwise specified in this permit or approved in writing by the Department of Ecology (Department).

D. Flow Measurement

Appropriate flow measurement devices and methods consistent with accepted scientific practices shall be selected and used to ensure the accuracy and reliability of measurements of the quantity of flows monitored as required in S1. The devices shall be installed, calibrated, and maintained to ensure that the accuracy of the measurements are consistent with the accepted industry standard for that type of device. Frequency of calibration shall be in conformance with manufacturer's recommendations and at a minimum frequency of at least one calibration per year. Calibration records shall be maintained for at least three (3) years.

E. Laboratory Accreditation

All monitoring data required by the Department shall be prepared by a laboratory registered or accredited under the provisions of, *Accreditation of Environmental Laboratories*, Chapter 173-50 WAC. Flow, temperature, settleable solids, conductivity, and internal process control parameters are exempt from this requirement. Crops, soils, and hazardous waste data are exempted from this requirement pending accreditation of laboratories for analysis of these media by the Department.

F. Metals Sampling and Analysis Report

By January 1, 2002, the Permittee shall submit a report to the Department on quality assurance and quality control measures for sampling and analyzing the concentrations of metals in the effluent. The report shall provide details on sampling equipment, sampling procedures, and laboratory analysis methods to minimize contamination of samples collected to meet the monitoring requirements for copper, zinc, and other priority pollutant metals. These measures shall be instituted for all sampling and analysis related to reporting the concentrations of metals in the effluent required by this permit.

S3. REPORTING AND RECORDKEEPING REQUIREMENTS

The Permittee shall submit reports, keep records, and provide notification in accordance with the following conditions. The falsification of information submitted to the Department shall constitute a violation of the terms and conditions of this permit.

A. Reporting

The first monitoring period begins on the effective date of the permit. Monitoring results shall be submitted monthly. Monitoring data obtained during the previous month shall be summarized and reported on forms provided by or approved by the Department (the Department's monthly report form and EPA form No. 3320-1). Submit reports so that they arrive at the Department's office no later than the 15th day of the month following the completed monitoring period, unless otherwise specified in this permit. The reports shall be sent to the Department of Ecology, 3190 – 160th Avenue SE, Bellevue, Washington 98008-5452.

Priority pollutant, copper, and zinc analysis data shall be submitted no later than forty-five (45) days following the monitoring period.

All lab reports providing data for organic and metal parameters shall include the following information: sampling date, sample location, date of analysis, parameter name, CAS number, analytical method/number, method detection limit (MDL), lab practical quantitation limit (PQL), reporting units and concentration detected.

B. Reporting Receiving Water Flows

From August 1 to November 15 the Permittee shall record and report the average daily flow of the Skagit River as measured and reported for US Geological Survey (USGS) gauging station number 12200500 named "Skagit River near Mount Vernon" and listed by USGS as located at Latitude 48° 26' 42" and Longitude 122° 20' 03". The flow shall be reported on the DMR forms provided by the Department. This information is available on the World Wide Web at http://wwwdwatcm.wr.usgs.gov/rt-cgi/gen_stn_pg?station=12200500.

In the event that USGS fails to record or report this flow, the Permittee is not required to report the flow so long as the Permittee notifies the Department within five (5) days that the flow data is not available.

The Permittee may propose alternative methodologies for obtaining the flows for obtaining and reporting Skagit River flows during the low flow months. These proposals shall be submitted in writing for approval by the Department.

C. Records Retention

The Permittee shall retain records of all monitoring information for a minimum of three (3) years. Such information shall include all calibration and maintenance records and all original recordings for continuous monitoring instrumentation, copies of all reports required by this permit, and records of all data used to complete the application for this permit. This period of retention shall be extended during the course of any unresolved litigation regarding the discharge of pollutants by the Permittee or when requested by the Director.

D. Recording of Results

For each measurement or sample taken, the Permittee shall record the following information: (1) the date, exact place, method, and time of sampling; (2) the individual who performed the sampling or measurement; (3) the dates the analyses were performed; (4) who performed the analyses; (5) the analytical techniques or methods used; and (6) the results of all analyses.

E. Additional Monitoring by the Permittee

If the Permittee monitors any pollutant more frequently than required by this permit using test procedures specified in this permit, then the results of this monitoring shall be included in the calculation and reporting of the data submitted in the Permittee's self-monitoring reports.

F. Noncompliance Notification

In the event the Permittee is unable to comply with any of the permit terms and conditions due to any cause, the Permittee shall:

1. Immediately take action to stop, contain, and cleanup unauthorized discharges or otherwise stop the violation, and correct the problem;

2. Repeat sampling and analysis of any violation and submit the results to the Department within thirty (30) days after becoming aware of the violation;

3. Immediately notify the Department of the failure to comply; and

4. Submit a detailed, written report to the Department within thirty (30) days (five [5] days for upsets and bypasses), unless requested earlier by the Department. The report should describe the nature of the violation, corrective action taken and/or planned, steps to be taken to prevent a recurrence, results of the re-sampling, and any other pertinent information.

Compliance with these requirements does not relieve the Permittee from responsibility to maintain continuous compliance with the terms and conditions of this permit or the resulting liability for failure to comply.

G. Compliance Progress Report

By April 1 of each year the Permittee shall submit an annual report covering for the previous year the progress made towards meeting the final effluent limits for copper and pH. The report should be limited to describing major milestones such as progress on or completion of facility plans, construction plans, or actual construction. This requirement shall lapse when the Permittee informs the Department that all final limits in the permit can be complied with.

S4. FACILITY LOADING

A. Design Criteria

The wet season design criteria for the permitted treatment facility are as follows:

Average flow for the maximum month	5.6 MGD
Influent BOD_5 loading for maximum month	8130 lbs./day
Influent TSS loading for maximum month	7181 lbs./day

The dry season design criteria for the permitted treatment facility are as follows:

Average flow for the maximum month	4.8 MGD
Influent BOD_5 loading for maximum month	8922 lbs./day
Influent TSS loading for maximum month	7181 lbs./day

B. Plans for Maintaining Adequate Capacity

When the actual flow or waste load reaches 85 percent of any one of the design criteria in S4.A for three (3) consecutive months, projected increases would reach design capacity within five (5) years, the Permittee shall submit a plan and schedule for continuing to maintain capacity at the facility sufficient to achieve the effluent limitations and other conditions of this permit. This plan shall address any of the following actions or any others necessary to meet this objective.

1. Analysis of the present design including the introduction of any process modifications that would establish the ability of the existing facility to achieve the effluent limits and other requirements of this permit at specific levels in excess of the existing design criteria specified in paragraph A above.

2. Reduction or elimination of excessive infiltration and inflow of uncontaminated ground and surface water into the sewer system.

3. Limitation on future sewer extensions or connections or additional waste load.

4. Modification or expansion of facilities necessary to accommodate increased flow or wasteland.

5. Reduction of industrial or commercial flows or waste loads to allow for increasing sanitary flow or waste load.

Engineering documents associated with the plan must meet the requirements of WAC 173-240-060, "Engineering Report," and be approved by the Department prior to any construction. The plan shall specify any contracts, ordinances, methods for financing, or other arrangements necessary to achieve this objective.

C. Notification of New or Altered Sources

The Permittee shall submit written notice to the Department whenever any new discharge or increase in volume or change in character of an existing discharge into the sewer is proposed which: (1) would interfere with the operation of, or exceed the design capacity of, any portion of the collection or treatment system; (2) is not part of an approved general sewer plan or approved plans and specifications; or would be subject to pretreatment standards under 40 CFR Part 403 and Section 307(b) of the Clean Water Act. This notice shall include an evaluation of the system's ability to adequately transport and treat the added flow and/or waste load.

S5. OPERATION AND MAINTENANCE

The Permittee shall at all times be responsible for the proper operation and maintenance of any facilities or systems of control installed to achieve compliance with the terms and conditions of the permit.

A. Certified Operator

An operator certified for at least a Class 3 plant by the State of Washington shall be in responsible charge of the day-to-day operation of the wastewater treatment plant. An operator certified for at least a Class 2 plant shall be in charge during all regularly scheduled shifts.

B. O & M Program

The Permittee shall institute an adequate operation and maintenance program for their entire sewage system. Maintenance records shall be maintained on all major electrical and mechanical components of the treatment plant, as well as the sewage system and pumping stations. Such records shall clearly specify the frequency and type of maintenance recommended by the manufacturer and shall show the frequency and type of maintenance performed. These maintenance records shall be available for inspection at all times.

C. Short-term Reduction

If a Permittee contemplates a reduction in the level of treatment that would cause a violation of permit discharge limitations on a short-term basis for any reason, and such reduction cannot be avoided, the Permittee shall give written notification to the Department, if possible, thirty (30) days prior to such activities, detailing the reasons for, length of time of, and the potential effects of the reduced level of treatment. This notification does not relieve the Permittee of their obligations under this permit.

D. Electrical Power Failure

The Permittee is responsible for maintaining adequate safeguards to prevent the discharge of untreated wastes or wastes not treated in accordance with the requirements of this permit during electrical power failure at the treatment plant and/or sewage lift stations either by means of alternate power sources, standby generator, or retention of inadequately treated wastes. The Permittee shall maintain Reliability Class II (EPA 430-99-74-001) at the wastewater treatment plant, which requires primary sedimentation and disinfection.

E. Prevent Connection of Inflow

The Permittee shall strictly enforce their sewer ordinances and not allow the connection of inflow (roof drains, foundation drains, etc.) to the sanitary sewer system in those areas where connection to the storm drain system are available.

F. Bypass Procedures

The Permittee shall immediately notify the Department of any spill, overflow, or bypass from any portion of the collection or treatment system.

The bypass of wastes from any portion of the treatment system is prohibited unless one of the following conditions (1, 2, or 3) applies:

1. Unavoidable Bypass -- Bypass is unavoidable to prevent loss of life, personal injury, or severe property damage. "Severe property damage" means substantial physical damage to property, damage to the treatment facilities which would cause them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of a bypass.

   If the resulting bypass from any portion of the treatment system results in noncompliance with this permit, the Permittee shall notify the Department in accordance with condition S3.E "Noncompliance Notification."

2. Anticipated Bypass That Has the Potential to Violate Permit Limits or Conditions -- Bypass is authorized by an administrative order issued by the Department. The Permittee shall notify the Department at least thirty (30) days before the planned date of bypass. The notice shall contain: (1) a description of the bypass and its cause; (2) an analysis of all known alternatives which would eliminate, reduce, or mitigate the need for bypassing; (3) a cost-effectiveness analysis of alternatives including comparative resource damage assessment; (4) the minimum and maximum duration of bypass under each alternative; (5) a recommendation as to the preferred alternative for conducting the bypass; (6) the projected date of bypass initiation; (7) a statement of compliance with SEPA; (8) if a water quality criteria exceedance is unavoidable, a request for modification of water quality standards as provided for in WAC 173-201A-110, and (9) steps taken or planned to reduce, eliminate, and prevent reoccurrence of the bypass.

For probable construction bypasses, the need to bypass is to be identified as early in the planning process as possible. The analysis required above shall be considered during preparation of the engineering report or facilities plan and plans and specifications and shall be included to the extent practical. In cases where the probable need to bypass is determined early, continued analysis is necessary up to and including the construction period in an effort to minimize or eliminate the bypass.

The Department will consider the following prior to issuing an administrative order:

a. If the bypass is necessary to perform construction or maintenance-related activities essential to meet the requirements of the permit.

b. If there are feasible alternatives to bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, maintenance during normal periods of equipment down time, or transport of untreated wastes to another treatment facility.

c. If the bypass is planned and scheduled to minimize adverse effects on the public and the environment.

After consideration of the above and the adverse effects of the proposed bypass and any other relevant factors, the Department will approve or deny the request. The public shall be notified and given an opportunity to comment on bypass incidents of significant duration, to the extent feasible. Approval of a request to bypass will be by administrative order issued by the Department under RCW 90.48.120.

3. Bypass For Essential Maintenance Without the Potential to Cause Violation of Permit Limits or Conditions -- Bypass is authorized if it is for essential maintenance and does not have the potential to cause violations of limitations or other conditions of the permit, or adversely impact public health as determined by the Department prior to the bypass.

G. Operations and Maintenance Manual

The approved Operations and Maintenance Manual (O&M) shall be kept available at the treatment plant. The operation and maintenance manual shall contain the plant process control monitoring schedule. The Permittee shall update the manual for new procedures and equipment as appropriate.

S6. PRETREATMENT

A. General Requirements

The Permittee shall work cooperatively with the Department to ensure that all commercial and industrial users of the wastewater treatment system are in compliance with the pretreatment regulations promulgated in 40 CFR Part 403 and any additional pretreatment regulations that may be promulgated under Section 307(b) and reporting requirements under Section 308 of the Federal Clean Water Act.

B. Discharge Authorization Required

Significant commercial or industrial operations shall not be allowed to discharge wastes to the Permittee's sewerage system until they have received prior authorization from the Department in accordance with chapter 90.48 RCW and chapter 173-216 WAC, as amended. The Permittee shall immediately notify the Department of any proposed new sources, as defined in 40 CFR 403.3(k), from significant commercial or industrial operations.

C. Prohibited Discharges

In accordance with 40 CFR 403.5(a), a nondomestic discharger may not introduce into the Permittee's sewerage system any pollutant(s) that cause pass through or interference.

D. Specific Prohibitions

In accordance with 40 CFR 403.5(b), the following nondomestic discharges shall not be discharged into the Permittee's sewerage treatment system.

1. Pollutants that create a fire or explosion hazard in the POTW (including, but not limited to waste streams with a closed cup flashpoint of less than 140 degrees Fahrenheit or 60 degrees Centigrade using the test methods specified in 40 CFR 261.21).

2. Pollutants that will cause corrosive structural damage to the Publicly Owned Treatment Works (POTW), but in no case discharges with pH lower than 5.0 standard units, unless the works are specifically designed to accommodate such discharges.

3. Solid or viscous pollutants in amounts that could cause obstruction to the flow in sewers or otherwise interfere with the operation of the POTW.

4. Any pollutant, including oxygen-demanding pollutants, (BOD, etc.) released in a discharge at a flow rate and/or pollutant concentration which will cause interference with the POTW.

5. Heat in amounts that will inhibit biological activity in the POTW resulting in interference, but in no case heat in such quantities such that the temperature at the POTW exceeds 40° C (104° F) unless the Department, upon request of the Permittee, approves, in writing, alternate temperature limits.

6. Petroleum oil, nonbiodegradable cutting oil, or products of mineral origin in amounts that will cause interference or pass through.

7. Pollutants which result in the presence of toxic gases, vapors, or fumes within the POTW in a quantity which may cause acute worker health and safety problems.

8. Any trucked or hauled pollutants, except at discharge points designated by the Permittee.

9. Wastewater prohibited to be discharged to the POTW by the Dangerous Waste Regulations (Chapter 173-303 WAC), unless authorized under the Domestic Sewage Exclusion (WAC 173-303-071).

10. All of the following are prohibited from discharge to the POTW unless approved in writing by the Department:

    a. Non-contact cooling water in significant volumes.

    b. Stormwater, and other direct inflow sources.

    c. Wastewater significantly affecting system hydraulic loading, which do not require treatment, or would not be afforded a significant degree of treatment by the system.

E. <u>Notification of Industrial User Violations</u>

The Permittee shall notify the Department in writing if any nondomestic user violates the prohibitions listed in S7.C and S7.D above.

F. Industrial User Survey

The Permittee shall prepare a list of industrial users to be submitted with the NPDES application required by Condition G.7. The list shall include any facility that meets any of the following criteria:

1. Discharges wastewater from a process regulated under the Pretreatment Categorical Standards contained in Subchapter N of Chapter I, Part 40 of the Code of Federal Regulations. Examples of categorical industrial users include electroplaters, anodizers, conversion coaters, electrical component manufacturers, printed circuit board manufacturers, semi-conductor manufacturers, chemical formulators/manufacturers, pharmaceutical manufacturers, soap and detergent manufacturers, paperboard manufacturers, petroleum refiners, iron and steel manufacturers, battery manufacturers, non-ferrous metal manufacturers, ferroalloy manufacturers, porcelain enamelers, steam electric generators, leather tanneries, coil coaters, foundries, aluminum formers, copper formers, and electrical component manufacturers.
2. Discharges 25,000 gpd or more of process wastewater to the sewer system.
3. Discharges priority or toxic pollutants in significant amounts.
4. Industrial food processors and agricultural product dischargers with significant loading to the treatment plant.

For each discharger listed, the Permittee will specify the volume of the discharge and the pollutants or types of chemicals contained in the discharge.

The Department may require the Permittee to perform other activities (e.g., sewer use ordinance and local limits development), which are necessary for the proper administration of the state pretreatment program.

S7. RESIDUAL SOLIDS

Residual solids include screenings, grit, scum, primary sludge, waste activated sludge and other solid waste. The Permittee shall store and handle all residual solids in such a manner so as to prevent their entry into state ground or surface waters. The Permittee shall not discharge leachate from residual solids to state surface or ground waters.

S8. ACUTE TOXICITY TESTING

A. Effluent Characterization

The Permittee shall conduct acute toxicity testing on the final effluent to determine the presence and amount of acute (lethal) toxicity. The two acute toxicity tests listed below shall be conducted on each sample taken for effluent characterization.

Effluent characterization for acute toxicity shall be conducted quarterly for one (1) year. Acute toxicity testing shall follow protocols, monitoring requirements, and quality assurance/quality control procedures specified in this section. A dilution series consisting of a minimum of five concentrations (including the ACEC of 20% effluent) and a control shall be used to estimate the concentration lethal to 50% of the organisms (LC_{50}). The percent survival in 100% effluent shall also be reported.

Testing shall begin by October 30, 2001. A written report shall be submitted to the Department within sixty (60) days after the sample date.

Acute toxicity tests shall be conducted with the following species and protocols:

1. Fathead minnow, Pimephales promelas (96-hour static-renewal test, method: EPA/600/4-90/027F).

2. Daphnid, Ceriodaphnia dubia, Daphnia pulex, or Daphnia magna (48-hour static test, method: EPA/600/4-90/027F). The Permittee shall choose one of the three species and use it consistently throughout effluent characterization.

B. <u>Effluent Limit for Acute Toxicity</u>

The Permittee has an effluent limit for acute toxicity if, after completing one year of effluent characterization, either:

(1) The median survival of any species in 100% effluent is below 80%, or

(2) Any one test of any species exhibits less than 65% survival in 100% effluent.

If an effluent limit for acute toxicity is required by subsection B at the end of one year of effluent characterization, the Permittee shall immediately complete all applicable requirements in subsections C, D, and F.

If no effluent limit is required by subsection B at the end of one year of effluent characterization, then the Permittee shall complete all applicable requirements in subsections E and F.

The ACEC means the maximum concentration of effluent during critical conditions at the boundary of the zone of acute criteria exceedance assigned pursuant to WAC 173-201A-100. **The effluent limit for acute toxicity is no acute toxicity detected in a test concentration of 20% effluent, which represents the acute critical effluent concentration (ACEC).**

In the event of failure to pass the test described in subsection C of this section for compliance with the effluent limit for acute toxicity, the Permittee is considered to be in compliance with all permit requirements for acute whole effluent toxicity as long as the requirements in subsection D are being met to the satisfaction of the Department.

If no effluent limit is required by subsection B at the end of one year of effluent characterization, then the Permittee shall stop effluent characterization and begin to conduct the activities in subsection E.

C. Monitoring for Compliance With an Effluent Limit for Acute Toxicity

Monitoring to determine compliance with the effluent limit shall be conducted quarterly for the remainder of the permit term using each of the species listed in subsection A on a rotating basis and performed using at a minimum 100% effluent, the ACEC of 20% effluent, and a control. The Permittee shall schedule the toxicity tests in the order listed in the permit unless the Department notifies the Permittee in writing of another species rotation schedule. The percent survival in 100% effluent shall be reported for all compliance monitoring.

Compliance with the effluent limit for acute toxicity means no statistically significant difference in survival between the control and the test concentration representing the ACEC. The Permittee shall immediately implement subsection D if any acute toxicity test conducted for compliance monitoring determines a statistically significant difference in survival between the control and the ACEC using hypothesis testing at the 0.05 level of significance (Appendix H, EPA/600/4-89/001). If the difference in survival between the control and the ACEC is less than 10%, the hypothesis test shall be conducted at the 0.01 level of significance.

D. Response to Noncompliance With an Effluent Limit for Acute Toxicity

If the Permittee violates the acute toxicity limit in subsection B, the Permittee shall begin additional compliance monitoring within one week from the time of receiving the test results. This additional monitoring shall be conducted weekly for four consecutive weeks using the same test and species as the failed compliance test. Testing shall determine the LC_{50} and effluent limit compliance. The discharger shall return to the original monitoring frequency in subsection C after completion of the additional compliance monitoring.

If the Permittee believes that a test indicating noncompliance will be identified by the Department as an anomalous test result, the Permittee may notify the Department that the compliance test result might be anomalous and that the Permittee intends to take only one additional sample for toxicity testing and wait for notification from the Department before completing the additional monitoring required in this subsection. The notification to the Department shall accompany the report of the compliance test result and identify the reason for considering the compliance test result to be anomalous. The Permittee shall complete all of the additional monitoring required in this subsection as soon as possible after notification by the Department that the compliance test result was not anomalous. If the one additional sample fails to comply with the effluent limit for acute toxicity, then the Permittee shall proceed without delay to complete all of the additional monitoring required in this subsection. The one additional test result shall replace the compliance test result upon determination by the Department that the compliance test result was anomalous.

If all of the additional compliance monitoring conducted in accordance with this subsection complies with the permit limit, the Permittee shall search all pertinent and recent facility records (operating records, monitoring results, inspection records, spill reports, weather records, production records, raw material purchases, pretreatment records, etc.) and submit a report to the Department on possible causes and preventive measures for the transient toxicity event which triggered the additional compliance monitoring.

If toxicity occurs in violation of the acute toxicity limit during the additional compliance monitoring, the Permittee shall submit a Toxicity Identification/Reduction Evaluation (TI/RE) plan to the Department. The TI/RE plan submittal shall be within sixty (60) days after the sample date for the fourth additional compliance monitoring test. If the Permittee decides to forgo the rest of the additional compliance monitoring tests required in this subsection because one of the first three additional compliance monitoring tests failed to meet the acute toxicity limit, then the Permittee shall submit the TI/RE plan within sixty (60) after the sample date for the first additional monitoring test to violate the acute toxicity limit. The TI/RE plan shall be based on WAC 173-205-100(2) and shall be implemented in accordance with WAC 173-205-100(3).

E. Monitoring When There Is No Permit Limit for Acute Toxicity

Recent WET testing results are required as part of the permit application. WET Tests that are less than two years old based on the sample collection date at the time the next permit application is due at the Department shall be considered recent. If the Permittee lacks two recent WET test results at the time the permit application (Condition G7), then they shall test final effluent at six month intervals prior to submission of the application for permit renewal in order to provide recent test results. All species used in the initial acute effluent characterization or substitutes approved by the Department shall be used and results submitted to the Department as a part of the permit renewal application process.

F. Sampling and Reporting Requirements

1. All reports for effluent characterization or compliance monitoring shall be submitted in accordance with the most recent version of Department of Ecology Publication #WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria* in regards to format and content. Reports shall contain bench sheets and reference toxicant results for test methods. If the lab provides the toxicity test data on floppy disk for electronic entry into the Department's database, then the Permittee shall send the disk to the Department along with the test report, bench sheets, and reference toxicant results.

2. Testing shall be conducted on 24-hour composite effluent samples. Composite samples taken for toxicity testing shall be cooled to 4 degrees Celsius while being collected and shall be sent to the lab immediately upon completion. Samples must be below 8° C at receipt. The lab shall begin the toxicity testing as soon as possible but no later than 36 hours after sampling was ended. The lab shall store all samples at 4° C in the dark from receipt until completion of the test.

3. All samples and test solutions for toxicity testing shall have water quality measurements as specified in Department of Ecology Publication #WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria* or most recent version thereof.

4. All toxicity tests shall meet quality assurance criteria and test conditions in the most recent versions of the EPA manual listed in subsection A and the Department of Ecology Publication #WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria*. If test results are determined to be invalid or anomalous by the Department, testing shall be repeated with freshly collected effluent.

5. Control water and dilution water shall be laboratory water meeting the requirements of the EPA manual listed in subsection A or pristine natural water of sufficient quality for good control performance.

6. The whole effluent toxicity tests shall be run on an unmodified sample of final effluent.

7. The Permittee may choose to conduct a full dilution series test during compliance monitoring in order to determine dose response. In this case, the series must have a minimum of five effluent concentrations and a control. The series of concentrations must include the ACEC of 20% effluent.

8. All whole effluent toxicity tests, effluent screening tests, and rapid screening tests that involve hypothesis testing, and do not comply with the acute statistical power standard of 29% as defined in WAC 173-205-020, must be repeated on a fresh sample with an increased number of replicates to increase the power.

S9. CHRONIC TOXICITY TESTING

A. Effluent Characterization

The Permittee shall conduct chronic toxicity testing on the final effluent. The two chronic toxicity tests listed below shall be conducted on each sample taken for effluent characterization.

Freshwater Chronic Toxicity Test Species		Method
Fathead minnow	*Pimephales promelas*	EPA/600/4-91/002
Water flea	*Ceriodaphnia dubia*	EPA/600/4-91/002

The first test shall begin by October 30, 2001, and the second test shall be conducted in the first half of 2002. A written report shall be submitted to the Department within sixty (60) days after the sample date for each test.

The Permittee shall conduct chronic toxicity testing during effluent characterization on a series of at least five concentrations of effluent in order to determine appropriate point estimates. This series of dilutions shall include the ACEC of 20% effluent and the CCEC of 3% effluent. The Permittee shall compare the ACEC to the control using hypothesis testing at the 0.05 level of significance as described in Appendix H, EPA/600/4-89/001.

B. <u>Effluent Limit for Chronic Toxicity</u>

After completion of effluent characterization, the Permittee has an effluent limit for chronic toxicity if any test conducted for effluent characterization shows a significant difference between the control and the ACEC of 20% effluent at the 0.05 level of significance using hypothesis testing (Appendix H, EPA/600/4-89/001) and shall complete all applicable requirements in subsections C, D, and F.

If no significant difference is shown between the ACEC of 20% effluent and the control in any of the chronic toxicity tests, the Permittee has no effluent limit for chronic toxicity and only subsections E and F apply.

The effluent limit for chronic toxicity is no toxicity detected in a test concentration representing the chronic critical effluent concentration of 3% effluent (CCEC).

In the event of failure to pass the test described in subsection C. of this section for compliance with the effluent limit for chronic toxicity, the Permittee is considered to be in compliance with all permit requirements for chronic whole effluent toxicity as long as the requirements in subsection D are being met to the satisfaction of the Department.

The CCEC means the maximum concentration of effluent allowable at the boundary of the mixing zone assigned in Condition S2 pursuant to WAC 173-201A-100. The CCEC equals 3% effluent.

C. Monitoring for Compliance With an Effluent Limit for Chronic Toxicity

Monitoring to determine compliance with the effluent limit shall be conducted biannually for the remainder of the permit term using each of the species listed in subsection A above on a rotating basis and performed using at a minimum the CCEC, the ACEC, and a control. The Permittee shall schedule the toxicity tests in the order listed in the permit unless the Department notifies the Permittee in writing of another species rotation schedule.

Compliance with the effluent limit for chronic toxicity means no statistically significant difference in response between the control and the test concentration representing the CCEC. The Permittee shall immediately implement subsection D. if any chronic toxicity test conducted for compliance monitoring determines a statistically significant difference in response between the control and the CCEC using hypothesis testing at the 0.05 level of significance (Appendix H, EPA/600/4-89/001). If the difference in response between the control and the CCEC is less than 20%, the hypothesis test shall be conducted at the 0.01 level of significance.

In order to establish whether the chronic toxicity limit is eligible for removal from future permits, the Permittee shall also conduct this same hypothesis test (Appendix H, EPA/600/4-89/001) to determine if a statistically significant difference in response exists between the ACEC and the control.

D. Response to Noncompliance With an Effluent Limit for Chronic Toxicity

If a toxicity test conducted for compliance monitoring under subsection C. determines a statistically significant difference in response between the CCEC and the control, the Permittee shall begin additional compliance monitoring within one (1) week from the time of receiving the test results. This additional monitoring shall be conducted monthly for three (3) consecutive months using the same test and species as the failed compliance test. Testing shall be conducted using a series of at least five effluent concentrations and a control in order to be able to determine appropriate point estimates. One of these effluent concentrations shall equal the CCEC and be compared statistically to the nontoxic control in order to determine compliance with the effluent limit for chronic toxicity as described in subsection C. The discharger shall return to the original monitoring frequency in subsection C after completion of the additional compliance monitoring.

If the Permittee believes that a test indicating noncompliance will be identified by the Department as an anomalous test result, the Permittee may notify the Department that the compliance test result might be anomalous and that the Permittee intends to take only one additional sample for toxicity testing and wait for notification from the Department before completing the additional monitoring required in this subsection. The notification to the Department shall accompany the report of the compliance test result and identify the reason for considering the compliance test result to be anomalous. The Permittee shall complete all of the additional monitoring required in this subsection as soon as possible after notification by the Department that the compliance test result was not anomalous. If the one additional sample fails to comply with the effluent limit for chronic toxicity, then the Permittee shall proceed without delay to complete all of the additional monitoring required in this subsection. The one additional test result shall replace the compliance test result upon determination by the Department that the compliance test result was anomalous.

If all of the additional compliance monitoring conducted in accordance with this subsection complies with the permit limit, the Permittee shall search all pertinent and recent facility records (operating records, monitoring results, inspection records, spill reports, weather records, production records, raw material purchases, pretreatment records, etc.) and submit a report to the Department on possible causes and preventive measures for the transient toxicity event which triggered the additional compliance monitoring.

If toxicity occurs in violation of the chronic toxicity limit during the additional compliance monitoring, the Permittee shall submit a Toxicity Identification/Reduction Evaluation (TI/RE) plan to the Department within sixty (60) days after test results are final. The TI/RE plan shall be based on WAC 173-205-100(2) and shall be implemented in accordance with WAC 173-205-100(3).

E. Monitoring When There Is No Permit Limit for Chronic Toxicity

Recent WET testing results are required as part of the permit application. WET Tests that are less than two years old based on the sample collection date at the time the next permit application is due at the Department shall be considered recent. If the Permittee lacks two recent WET test results at the time the permit application (Condition G7), then they shall test final effluent at six month intervals prior to submission of the application for permit renewal in order to provide recent test results. All species used in the initial chronic effluent characterization or substitutes approved by the Department shall be used and results submitted to the Department as a part of the permit renewal application process.

F. Sampling and Reporting Requirements

1. All reports for effluent characterization or compliance monitoring shall be submitted in accordance with the most recent version of Department of Ecology Publication # WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria,* in regards to format and content. Reports shall contain bench sheets and reference toxicant results for test methods. If the lab provides the toxicity test data on floppy disk for electronic entry into the Department's database, then the Permittee shall send the disk to the Department along with the test report, bench sheets, and reference toxicant results.

2. Testing shall be conducted on 24-hour composite effluent samples. Samples taken for toxicity testing shall be cooled to 4 degrees Celsius while being collected and shall be sent to the lab immediately upon completion. The lab shall begin the toxicity testing as soon as possible but no later than thirty-six (36) hours after sampling was ended.

3. All samples and test solutions for toxicity testing shall have water quality measurements as specified in Department of Ecology Publication # WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria,* or most recent version thereof.

4. All toxicity tests shall meet quality assurance criteria and test conditions in the most recent versions of the EPA manual listed in subsection A. and the Department of Ecology Publication # WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria*. If test results are determined to be invalid or anomalous by the Department, testing shall be repeated with freshly collected effluent.

5. Control water and dilution water shall be laboratory water meeting the requirements of the EPA manual listed in subsection A or pristine natural water of sufficient quality for good control performance.

6. The whole effluent toxicity tests shall be run on an unmodified sample of final effluent.

7. The Permittee may choose to conduct a full dilution series test during compliance monitoring in order to determine dose response. In this case, the series must have a minimum of five effluent concentrations and a control. The series of concentrations must include the ACEC and the CCEC.

8. All whole effluent toxicity tests, effluent screening tests, and rapid screening tests that involve hypothesis testing and do not comply with the chronic statistical power standard of 39% as defined in WAC 173-205-020 must be repeated on a fresh sample with an increased number of replicates to increase the power.

S10. COMBINED SEWER OVERFLOWS

A. Operations

The table below lists combined sewer overflows (CSOs) outfalls, from which a dilute mixture of untreated sewage and rainwater is discharged as a result of precipitation events. These outfalls are hydraulically connected and may discharge simultaneously. Discharges from these sites are prohibited except as a result of precipitation events. The Permittee shall minimize the amount of flow and levels of pollutants discharged at these locations by prudent operation of the CSO storage system and sewage treatment plant.

Each outfall shall have a sign posted advising the public of the potential discharge of untreated sewage and phone numbers of City representatives who will provide additional information about the CSOs.

OUTFALL	CSO DESIGNATION	LATITUDE	LONGITUDE
002	Park Street	48° 24' 50" N	122° 20' 15" W
003	Division Street	48° 25' 10" N	122° 20' 15" W

B. Combined Sewer Overflow Monitoring and Annual Report

The Permittee shall submit an annual CSO Report covering the previous calendar year to the Department by April 1 each year that complies with the requirements of WAC 173-245-090(1):

1. Detail the past year's frequency and volume of combined sewage discharged from each CSO discharge site.
2. Explain the CSO reduction accomplishments of the previous year.
3. List the projects associated with CSO reduction planned for the next year.
4. If there is an increase in the annual baseline volume or frequency of CSO discharges, then the City shall propose a project and schedule to reduce that CSO site or group of sites to or below the baseline condition.

At least once per year, the Permittee shall obtain composite samples that are representative of the CSO discharge(s) to estimate BOD_5 and TSS levels of those discharges at the three different overflow points in the CSO storage system. Sampling is only required for those overflow points that actually do overflow. At least once per year, the Permittee shall obtain grab samples from the Skagit River to estimate the fecal coliform bacteria levels upstream and downstream of the CSO outfalls. This sampling of the Skagit River shall only be required for overflow episodes that occur during daylight hours on weekdays. Results of this monitoring shall be submitted with the annual CSO report.

The Permittee shall maintain a current CSO sampling plan that explains how these monitoring and characterization requirements are met. Within three (3) months of the permit issuance date, the Permittee shall submit an update of the sampling plan that was submitted to the Department as part of the 1999 annual CSO report. The Permittee shall update or revise the sampling plan as necessary or if requested by the Department.

C. Combined Sewer Overflow Reduction Plan Amendment

In conjunction with the application for renewal of this permit, the Permittee shall submit an amendment of its CSO Reduction Plan to the Department for review. The amendment provide brief summaries of:

1. An assessment of the CSO reduction plan to date.
2. Changes or refinements to the original CSO reduction plan.
3. List of projects scheduled over the next five years that contribute to meeting the control mechanisms proposed in the CSO reduction plan.

D. Compliance Schedule

The Permittee shall continue with planning and engineering efforts to comply with the provisions of Order On Consent No. DE 96WQ-N105 and reduce the frequency of CSO events to an average of one per year no later than January 1, 2015.

GENERAL CONDITIONS

G1. SIGNATORY REQUIREMENTS

All applications, reports, or information submitted to the Department shall be signed and certified.

A. All permit applications shall be signed by either a principal executive officer or a ranking elected official.

B. All reports required by this permit and other information requested by the Department shall be signed by a person described above or by a duly authorized representative of that person. A person is a duly authorized representative only if:

1. The authorization is made in writing by a person described above and submitted to the Department, and

2. The authorization specifies either an individual or a position having responsibility for the overall operation of the regulated facility, such as the position of plant manager, superintendent, position of equivalent responsibility, or an individual or position having overall responsibility for environmental matters. (A duly authorized representative may thus be either a named individual or any individual occupying a named position.)

C. Changes to authorization. If an authorization under paragraph B.2. above is no longer accurate because a different individual or position has responsibility for the overall operation of the facility, a new authorization satisfying the requirements of B.2. must be submitted to the Department prior to or together with any reports, information, or applications to be signed by an authorized representative.

D. Certification. Any person signing a document under this section shall make the following certification:

"I certify under penalty of law, that this document and all attachments were prepared under my direction or supervision in accordance with a system designed to assure that qualified personnel properly gathered and evaluated the information submitted. Based on my inquiry of the person or persons who manage the system or those persons directly responsible for gathering information, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete. I am aware that there are significant penalties for submitting false information, including the possibility of fine and imprisonment for knowing violations."

G2. RIGHT OF ENTRY

The Permittee shall allow an authorized representative of the Department, upon the presentation of credentials and such other documents as may be required by law:

A. To enter upon the premises where a discharge is located or where any records must be kept under the terms and conditions of this permit;

B. To have access to and copy at reasonable times any records that must be kept under the terms of the permit;

C. To inspect at reasonable times any monitoring equipment or method of monitoring required in the permit;

D. To inspect at reasonable times any collection, treatment, pollution management, or discharge facilities; and

E. To sample at reasonable times any discharge of pollutants.

G3. PERMIT ACTIONS

This permit shall be subject to modification, suspension, or termination, in whole or in part by the Department for any of the following causes:

A. Violation of any permit term or condition;

B. Obtaining a permit by misrepresentation or failure to disclose all relevant facts;

C. A material change in quantity or type of waste disposal;

D. A material change in the condition of the waters of the state; or

E. Nonpayment of fees assessed pursuant to RCW 90.48.465.

The Department may also modify this permit, including the schedule of compliance or other conditions, if it determines good and valid cause exists, including promulgation or revisions of regulations or new information.

G4. REPORTING A CAUSE FOR MODIFICATION

The Permittee shall submit a new application, or a supplement to the previous application, along with required engineering plans and reports, whenever a material change in the quantity or type of discharge is anticipated which is not specifically authorized by this permit. This application shall be submitted at least sixty (60) days prior to any proposed changes. Submission of this application does not relieve the Permittee of the duty to comply with the existing permit until it is modified or reissued.

G5. PLAN REVIEW REQUIRED

Prior to constructing or modifying any wastewater control facilities, an engineering report and detailed plans and specifications shall be submitted to the Department for approval in accordance with Chapter 173-240 WAC. Engineering reports, plans, and specifications should be submitted at least one hundred and eighty (180) days prior to the planned start of construction. Facilities shall be constructed and operated in accordance with the approved plans.

G6. COMPLIANCE WITH OTHER LAWS AND STATUTES

Nothing in the permit shall be construed as excusing the Permittee from compliance with any applicable federal, state, or local statutes, ordinances, or regulations.

G7. DUTY TO REAPPLY

The Permittee must apply for permit renewal at least one hundred and eighty (180) days prior to the specified expiration date of this permit. A new authorization letter for signature authority as described in G1.B shall be submitted with the application.

G8. REMOVED SUBSTANCES

Collected screenings, grit, solids, sludge, filter backwash, or other pollutants removed in the course of treatment or control of wastewater shall not be re-suspended or reintroduced to the final effluent stream for discharge to state waters.

G9. TOXIC POLLUTANTS

If any applicable toxic effluent standard or prohibition (including any schedule of compliance specified in such effluent standard or prohibition) is established under Section 307(a) of the Clean Water Act for a toxic pollutant and that standard or prohibition is more stringent than any limitation upon such pollutant in the permit, the Department shall institute proceedings to modify or revoke and reissue the permit to conform to the new toxic effluent standard or prohibition.

G10. OTHER REQUIREMENTS OF 40 CFR

All other requirements of 40 CFR 122.41 and 122.42 are incorporated in this permit by reference.

G11. ADDITIONAL MONITORING

The Department may establish specific monitoring requirements in addition to those contained in this permit by administrative order or permit modification.

G12. PAYMENT OF FEES

The Permittee shall submit payment of fees associated with this permit as assessed by the Department. The Department may revoke this permit if the permit fees established under Chapter 173-224 WAC are not paid.

G13. PENALTIES FOR VIOLATING PERMIT CONDITIONS

Any person who is found guilty of willfully violating the terms and conditions of this permit shall be deemed guilty of a crime, and upon conviction thereof shall be punished by a fine of up to ten thousand dollars and costs of prosecution, or by imprisonment in the discretion of the court. Each day upon which a willful violation occurs may be deemed a separate and additional violation.

Any person who violates the terms and conditions of a waste discharge permit shall incur, in addition to any other penalty as provided by law, a civil penalty in the amount of up to ten thousand dollars for every such violation. Each and every such violation shall be a separate and distinct offense, and in case of a continuing violation, every day's continuance shall be and be deemed to be a separate and distinct violation.

FACT SHEET FOR NPDES PERMIT WA-002407-4

City of Mount Vernon Wastewater Treatment Plant

SUMMARY

Changes to the sewer system since 1993 and new information about the outfall and receiving water have all led to significant changes to the permit conditions. The amount of dilution estimated for the outfall has been reduced based on an outfall study provided by the Permittee. The Department finalized a TMDL study for the lower Skagit River to set maximum discharge levels for ammonia, BOD_5, and fecal coliform bacteria to the lower Skagit River. Combined sewer overflows (CSOs) have been reduced by about 90% by storing flows for treatment at the WWTP. The Department approved a revision to plant capacity. Chemical dechlorination was added to the treatment system to remove chlorine from the effluent. Planning mandated under the State's Growth Management Act has provided a basis for revising projected need for increased plant capacity. The City is preparing a new facilities plan to address the need for additional capacity and compliance with new water quality-based requirements.

The outfall study concluded that a new outfall is needed to provide increased future capacity, process treated combined sewage flows from the plant, and enhance the dilution of effluent in the immediate vicinity of the outfall.

New water quality-based limits for ammonia, pH, copper, and zinc are in the permit. The new copper and pH limits can not be met based on previous data collected at the plan, so the Department imposed interim limits for these pollutants. The permit requires a sampling plan for metals to ensure that data provided on metals concentrations is reliable. The plant has no facilities to adjust pH to meet the pH limit. The permit provides two (2) years to plan for how the copper and pH limits will be met.

The permit requires whole effluent toxicity testing to characterize the effluent to obtain current information using the standards in Ecology rules enacted since the last permit was issued.

The plant capacity has been revised based on actual operating conditions and stress testing at the plant. Flow capacity is increased by 40% to 5.6 MGD and organic capacity is reduced by 46% to 8130 lbs./day for the maximum month in the winter.

The permit incorporates the waste load allocations derived from the Lower Skagit Total Maximum Daily Load Submittal Report (Ecology, 2000) to maintain compliance with dissolved oxygen standards by limiting summertime discharge of pounds of ammonia. The Skagit River currently complies with dissolved oxygen standards.

Specific process control testing requirements have been removed; that testing is to be done per the plant operations and maintenance manual.

An industrial user survey is required with the next permit submittal.

CSO testing requirements are added – CSO overflows have been reduced significantly; sampling and flow measure capabilities for CSOs have been added to the control system. The requirement to remove 85% of influent BOD_5 and TSS concentration has been relaxed to 80% because transporting CSO flows to the plant reduces influent strength.

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INTRODUCTION

The Federal Clean Water Act (FCWA, 1972, and later modifications, 1977, 1981, and 1987) established water quality goals for the navigable (surface) waters of the United States. One of the mechanisms for achieving the goals of the Clean Water Act is the National Pollutant Discharge Elimination System of permits (NPDES permits), which is administered by the Environmental Protection Agency (EPA). The EPA has delegated responsibility to administer the NPDES permit program to the State of Washington on the basis of Chapter 90.48 RCW which defines the Department of Ecology's authority and obligations in administering the wastewater discharge permit program.

The regulations adopted by the State include procedures for issuing permits (Chapter 173-220 WAC), technical criteria for discharges from municipal wastewater treatment facilities (Chapter 173-221 WAC), water quality criteria for surface and ground waters (Chapters 173-201A and 200 WAC), and sediment management standards (Chapter 173-204 WAC). These regulations require that a permit be issued before discharge of wastewater to waters of the state is allowed. The regulations also establish the basis for effluent limitations and other requirements which are to be included in the permit. One of the requirements (WAC 173-220-060) for issuing a permit under the NPDES permit program is the preparation of a draft permit and an accompanying fact sheet. Public notice of the availability of the draft permit is required at least thirty (30) days before the permit is issued (WAC 173-220-050). The fact sheet and draft permit are available for review (see <u>Appendix A--Public Involvement</u> of the fact sheet for more detail on the public notice procedures).

The fact sheet and draft permit have been reviewed by the Permittee. Errors and omissions identified in this review have been corrected before going to public notice. After the public comment period has closed, the Department will summarize the substantive comments and the response to each comment. The summary and response to comments will become part of the file on the permit, and parties submitting comments will receive a copy of the Department's response. The fact sheet will not be revised. Comments and the resultant changes to the permit will be summarized in Appendix E--Response to Comments.

GENERAL INFORMATION	
Applicant	City of Mount Vernon
Facility Name and Address	City of Mount Vernon Wastewater Treatment Plant 1401 Britt Road Mount Vernon, Washington 98273
Responsible Official	The Honorable Skye Richendrfer - Mayor City of Mount Vernon P O B 809 Mount Vernon, WA 98273 phone 360-336-6219

GENERAL INFORMATION	
Facility Contacts	Walter Enquist – Superintendent, (360) 336-6219 Bill Fullner – Process Analyst, (360) 336-6219 fax: (360) 424-8749 John W. Buckley – Public Works Director, (360)336-6204
Type of Treatment	Secondary biological treatment (conventional mixed activated sludge, chlorine disinfection, chemical de-chlorination, anaerobic sludge digestion)
Plant Discharge Location – Outfall 1	Skagit River Latitude: 48° 28' 04" N Longitude: 122° 18' 30" W
CSO Discharge Locations – Outfall 2 (Park Street) – Outfall 3 (Division St.)	Skagit River Latitude: 48° 24' 50" N Longitude: 122° 20' 15" W Latitude: 48° 25' 10" N Longitude: 122° 20' 15" W
Water Body ID Number	WA-03-1010

BACKGROUND INFORMATION

DESCRIPTION OF THE FACILITY

HISTORY

The City of Mount Vernon's sewerage system dates from the early 1900's. Combined sanitary and storm sewers served the area of Mount Vernon constructed prior to about 1948; combined sewer discharges into the Skagit River occurred at the foot of Division and Park Streets. This waste flow was intercepted and diverted to a primary treatment plant constructed in 1948.

In the late 1960's, the State required an upgrade of the plant to provide secondary treatment. At the time there were no federal secondary treatment standards. Accordingly, a secondary plant was designed to achieve the State standard at that time of 85% BOD and 90% TSS removal. The plant configuration after a 1972 upgrade included a primary clarifier, an oxidation tower (biofilter), a secondary clarifier, an anaerobic digester, chlorine disinfection, and a sludge thickener. Construction of that facility was completed in 1974.

During the middle of the 1970's, the plant performed adequately and achieved the effluent limits specified in the City's NPDES permit. The population of the City increased by almost 50% during the 1970's. Industrial loading increased substantially. The quality of the effluent from the plant deteriorated as the influent BOD loading increased. By the early 1980's, the plant failed to meet effluent discharge standards because it was hydraulically overloaded. Mount Vernon had to upgrade the plant again. This expansion and upgrade of the WWTP was completed in the fall of 1989 to increase capacity and improve performance.

During the early 1990's, the City provided for meeting State regulations to reduce combined sewer overflows (CSO). In 1998, Mount Vernon put the Central CSO Regulator into service. This facility was constructed as the first phase of reducing CSO overflow events to an average of once per year. The CSO Regulator system is composed of one mile of 5-foot diameter sewer pipe around the old downtown area. The system stores wet weather flows for eventual discharge to the treatment plant and provides flow measurement and CSO sampling capability. This regulator was designed to reduce overflows by about 90% and provide design information to meet the once per year overflow standard by 2015.

The Department conducted a Total Maximum Daily Load study on the lower Skagit River during the 1990's. The results of the study provide limitations on the output of the plant to the river based on the Department's estimate of how much oxygen demanding pollutants can be discharged to the lower Skagit without violating state standards for dissolved oxygen in the lower river. The study results provide limits for $CBOD_5$ (or BOD_5) and ammonia needed to continue meeting state water quality standards for dissolved oxygen in the receiving water.

COLLECTION SYSTEM STATUS

The primary source of waste water tributary to the facility is domestic sewage from residential and light commercial activities in the City of Mount Vernon. The collection system tributary to the plant is composed of separate sanitary sewers and combined storm water and sanitary sewers (CSO) from the older sections of the town. The sewer system and pump stations are maintained and repaired promptly when breakdowns occur. The plant is directly influenced by precipitation events.

TREATMENT PROCESSES

The 1989 plant upgrade was based on a design population of 28,500. The 2000 Census lists the actual population as 26,300. The 1989 upgrade included the following additions: three aeration basins, an aerobic digester, a secondary clarifier, a dissolved-air flotation thickener, an anaerobic digester, and a belt filter press. The plant flow scheme also includes a bar screen, an influent pump station, and two comminutors prior to the primary clarifier. A flow chart for the plant processes is shown on the following page.

Influent sewage enters the plant below ground, and then is pumped up to the headworks. From there, sewage flows to the primary clarifiers where solids settle out. Then sewage flows to another set of basins where flows mix with a bacterial mass while under aeration (conventional mixed activated sludge treatment process). The mixed liquor flows from the aeration basins to the secondary clarifiers where the organic solids settle out. The secondary clarifier effluent is disinfected by injecting chlorine in a flash mixer. Chlorine is chemically removed with sodium bisulfite after disinfection. Disinfected effluent is discharged to the Skagit River via a submerged outfall pipe. During high river flows the effluent is pumped to the river.

Sludge generated during the treatment process is thickened by a dissolved-air flotation thickener and routed to the primary anaerobic digester. Digested sludge is further thickened in the supernatant tank and by belt presses. Methane gas is recovered from the anaerobic digestion tank and is used as a heating source. Digested sludge is applied to farmland in eastern Washington.

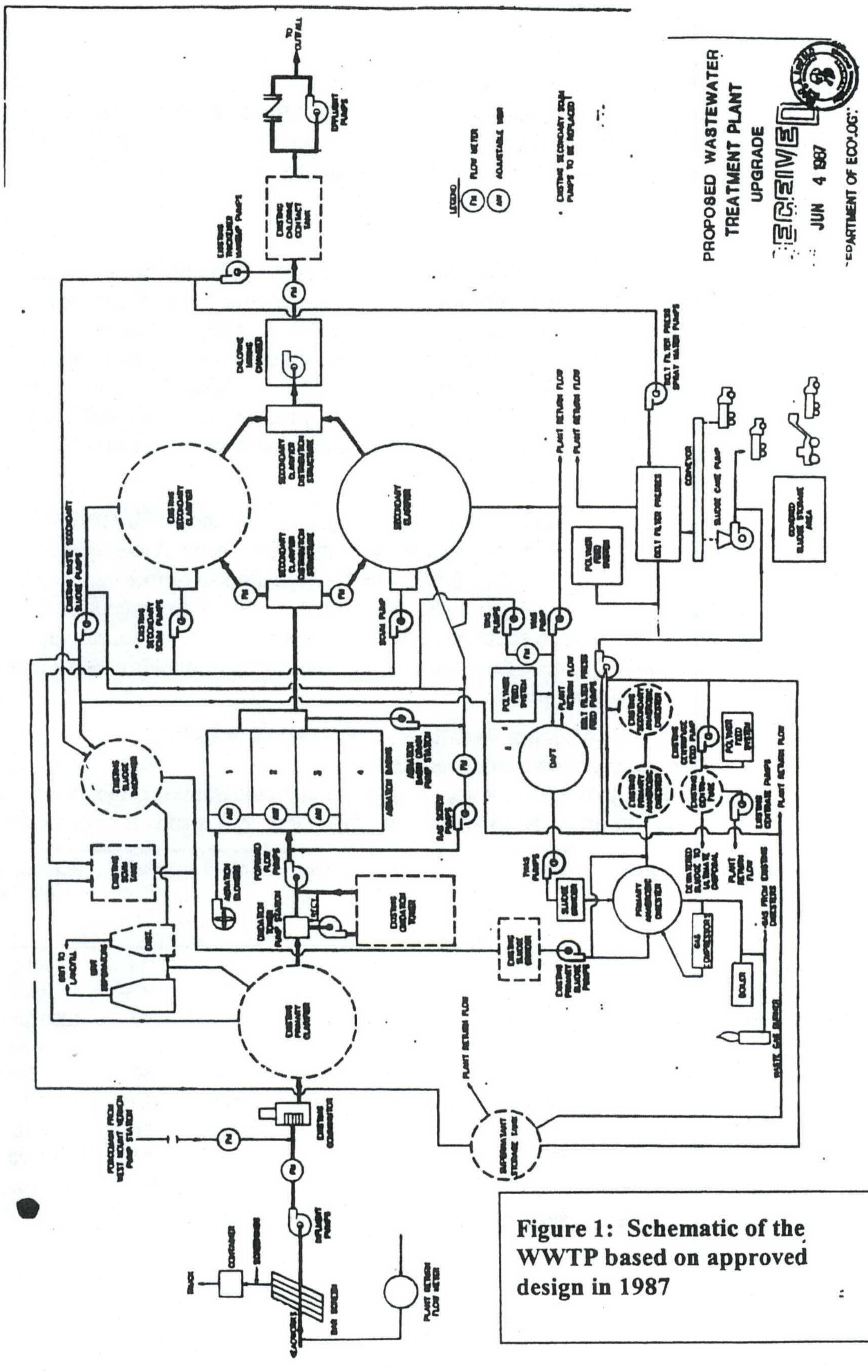

Figure 1: Schematic of the WWTP based on approved design in 1987

DISCHARGE OUTFALL

Secondary treated and disinfected effluent is discharged from the facility via a 24-inch ductile iron outfall pipe into the Skagit River. Two CSO outfalls are located upstream of the WWTP outfall, one at the Division Street Bridge about ¾ mile upstream of the WWTP outfall and the other near the end of Park Street less than ¼ mile upstream of the WWTP outfall.

RESIDUAL SOLIDS

This treatment facility removes solids at the headworks (screenings), in the primary and secondary clarifiers, and as part of the routine maintenance of the equipment (rags, scum, and other debris). The solids (the sludge without the debris) are processed into biosolids using anaerobic digestion (microbial digestion in large tanks for greater than 20 days) and are then pressed to remove water. The biosolids are hauled to Douglas County for land application on farm fields owned by Boulder Park, Inc. The biosolids are used as a soil amendment for growing crops. Grit, rags, scum, and screenings are drained and disposed of as solid waste at the Roosevelt landfill.

The facility monitors the quality of the biosolids in accordance with 40 CFR 503, the federal regulations for sludge processing and biosolids disposal. Annual averages of monitoring results for the period of 1993-99 are shown in Table 1. Pollutants are more concentrated in the sludge samples than in wastewater discharges. Numerical standards for exceptional quality biosolids are listed for comparison to actual results. Exceptional quality biosolids can be distributed to the public for garden use. These data demonstrate low levels of metals present in the biosolids produced at the treatment plant.

Table 1: Sludge Monitoring Results for 1994 Through 1999:
The values shown are annual averages.

Values marked with an *asterisk are averages of detected values, some of these values were below the detection limit. ND stands for not detected. NA stands for not analyzed.

All units are mg/Kg dry weight	1993	1994	1995	1996	1997	1998	1999	Except Qual Standard
Arsenic	ND	ND	ND	*3.8	*3.9	*3.3	*14	41
Cadmium	ND	*3	4.7	5.1	*3.6	*4	*3.3	39
Chromium	59	47	36	47	39	43	40	1200 (deleted)
Copper	443	385	317	348	255	267	309	1500
Lead	*84	*104	102	129	105	90	80	300
Mercury	3.4	*6	*2.2	2.2	1.9	*1.5	1.1	17
Molybdenum	NA	*12	*1.7	*5.5	8.9	*8.3	*7	18
Nickel	*61	*35	42	67	47	38	34	420
Selenium	ND	ND	ND	*7.6	*3.7	*7	*5.2	100
Zinc	623	533	599	747	593	576	574	2800

PERMIT STATUS

The previous permit for this facility was issued in June 1993 and expired in June 1998. The previous permit placed effluent limitations on 5-day Biochemical Oxygen Demand (BOD_5), Total Suspended Solids (TSS), pH, Fecal Coliform Bacteria, chlorine, and copper. The copper limit was removed from the permit in 1995 when the Department obtained new information that showed ambient levels of copper in the Skagit River were significantly lower than levels reported by USGS during the 1980's.

An application for permit renewal was submitted to the Department on December 16, 1997, and the permit was subsequently extended by the Department.

SUMMARY OF COMPLIANCE WITH THE PREVIOUS PERMIT

The facility received its last inspection on June 23, 1999; this was an inspection primarily focused on the CSO control system.

Based on Discharge Monitoring Reports (DMRs) submitted to the Department and inspections conducted by the Department during the last five years, the Permittee has complied with the permit conditions and effluent limitations except for the effluent violations listed in Table 2. The violations in January 1998 are due to processing high flows through the WWTP instead of discharging CSOs directly to the Skagit River. Initial operation of the CSO control system attempted to process more dilute influent than the plant was capable of sustaining. The violation in 1999 was due to high flows through the plant. The chlorine violation in 1995 was due to extended high flows through the WWTP over the entire month. Plant staff have worked diligently at meeting permit limitations and fixing any problems that occur.

Table 2: Violations of Permit Conditions.

parameter	duration	unit	reported value	limit	date
SOLIDS, TOTAL SUSPENDED	7-day average	LBS/DAY	1763	1501	Dec-99
BOD, 5-DAY (20 DEG. C)	30-day average	MG/L	36	30	Jan-98
SOLIDS, TOTAL SUSPENDED	30-day average	LBS/DAY	3222	1001	Jan-98
SOLIDS, SUSPENDED, % REMOVAL	30-day average	PERCENT	52	85	Jan-98
BOD, 5-DAY (20 DEG. C)	7-day average	LBS/DAY	4281	1501	Jan-98
BOD, 5-DAY (20 DEG. C)	30-day average	LBS/DAY	1851	1001	Jan-98
SOLIDS, TOTAL SUSPENDED	30-day average	MG/L	72	30	Jan-98
SOLIDS, TOTAL SUSPENDED	7-day average	MG/L	193	45	Jan-98
BOD, 5-DAY PERCENT REMOVAL	30-day average	PERCENT	73	85	Jan-98
BOD, 5-DAY (20 DEG. C)	7-day average	MG/L	70	45	Jan-98
SOLIDS, TOTAL SUSPENDED	7-day average	LBS/DAY	8896	1501	Jan-98
CHLORINE, TOTAL RESIDUAL	30-day average	LBS/DAY	19.2	15.6	Nov-95
CHLORINE, TOTAL RESIDUAL	30-day average	MG/L	0.5	0.47	Nov-95

The WWTP plant has also exceeded 85% of the facility design criteria listed in the previous permit on numerous instances over the last five years; these excursions are not permit violations.

In 1996, the City submitted an engineering analysis to the Department that revised the design criteria for the plant based on performance and stress testing.

WASTEWATER CHARACTERIZATION

The concentration of pollutants in the discharge was reported in the NPDES application and in discharge monitoring reports. The effluent is characterized as follows:

Table 3: Wastewater Characterization for 1999.

Parameter	Annual Average	Highest Monthly Average Value	Permit Limit
BOD_5	16 mg/L 530 lbs./day	23 mg/L 867 lbs./day	30 mg/L 1001 lbs./day
TSS	12 mg/L 446 lbs./day	20 mg/L 899 lbs./day	30 mg/L 1001 lbs./day
Fecal Coliform Bacteria	15 CFU/100mL (median)	93 CFU/100mL	200 CFU/100mL
pH varied from 6.9 to 7.0 standard units			

Influent pollutant loading and flow volume are important considerations for complying with secondary treatment standards at the WWTP. The permit requires that the City begin planning when either flows or waste loads reach 85% of the plant's capacity. The City submitted a re-evaluation of plant capacity in 1996 (RW Beck, 1995).

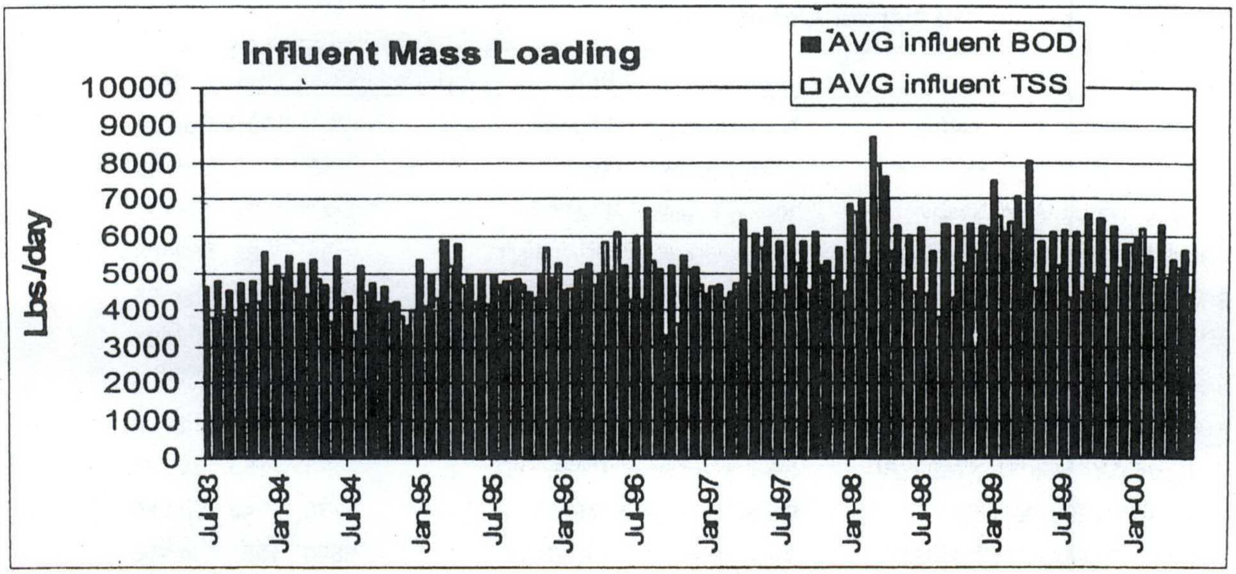

Figure 2: Influent Mass Loading.
The plant was originally rated with a capacity of 15,000 lbs./day of BOD and 7,800 lbs./day of TSS. The design capacity for BOD_5 loading has been recalcuated as 8,130 lbs./day for the wet season.

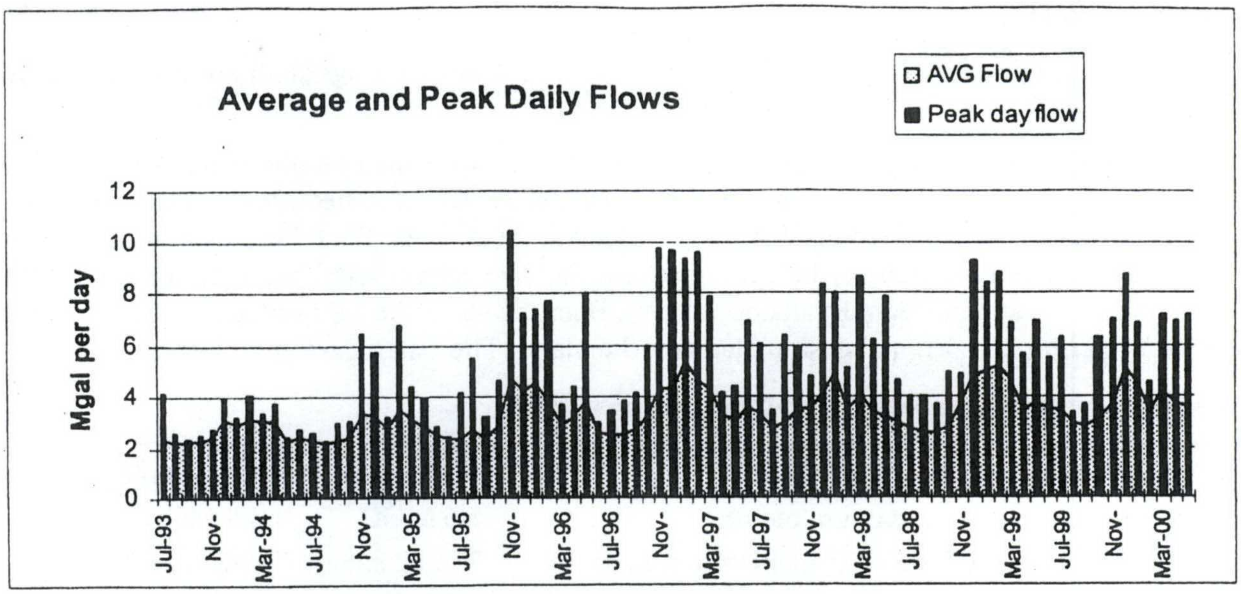

Figure 3: Average Monthly and Peak Daily Flows Through the WWTP. The original wet season design capacity was 4.0 MGD, the design capacity has been recalculated as 5.6 MGD.

The plant monitoring records indicate that the plant has not yet reached its design loading, but flows from December 1998 through February 1999 exceeded 85% of design (4.76 MGD average monthly flow) for three (3) consecutive months. The City is currently planning for treating increased flows as required by condition S4 of the permit.

PROPOSED PERMIT LIMITATIONS

Federal and State regulations require that effluent limitations set forth in a NPDES permit must be either technology- or water quality-based. Technology-based limitations for municipal discharges are set by regulation (40 CFR 133, and Chapters 173-220 and 173-221 WAC). Water quality-based limitations are based upon compliance with the Surface Water Quality Standards (Chapter 173-201A WAC), Ground Water Standards (Chapter 173-200 WAC), Sediment Quality Standards (Chapter 173-204 WAC), or the National Toxics Rule (Federal Register, Volume 57, No. 246, Tuesday, December 22, 1992.) The most stringent of these types of limits must be chosen for each of the parameters of concern. Each of these types of limits is described in more detail below.

The limits in this permit are based in part on information received in the application. The effluent constituents in the application were evaluated on a technology- and water quality-basis. The limits necessary to meet the rules and regulations of the State of Washington were determined and included in this permit. Ecology does not develop effluent limits for all pollutants that may be reported on the application as present in the effluent. Some pollutants are not treatable at the concentrations reported, are not controllable at the source, are not listed in regulation, and do not have a reasonable potential to cause a water quality violation. If significant changes occur in any constituent, as described in 40 CFR 122.42(a), the Permittee is required to notify the Department of Ecology.

DESIGN CRITERIA

In accordance with WAC 173-220-150 (1)(g), flows or waste loadings shall not exceed approved design criteria.

The original hydraulic and organic design criteria for this treatment facility were set based on the plant design as listed in the Contract No.4 Wastewater Facilities Project, Wastewater Treatment Plant for City of Mount Vernon, WA by RW Beck & Associates. That Facility Plan was approved by the Department on May 20, 1987. Those criteria have been revised based on the Mount Vernon Wastewater Treatment Plant Evaluation (R. W. Beck, 1995). The TSS criteria are taken from the previous permit and original design plans for the plant. The criteria are as follows:

Table 4: Design Standards for Mount Vernon WWTP.

Design Standards for wet months	**Wet season**	**Dry season**
Average Flow for the maximum month	5.6 MGD	4.8 MGD
Average Influent BOD_5 for the maximum month	8130 lbs./day	8920 lbs./day
Average Influent TSS for the maximum month	7800 lbs./day	7800 lbs./day

This plant has experienced a peak flow of 12 MGD during one CSO episode; effluent quality degraded to nearly beyond permit limits during that episode.

TECHNOLOGY-BASED EFFLUENT LIMITATIONS

Municipal wastewater treatment plants are a category of discharger for which technology-based effluent limits have been promulgated by federal and state regulations. These effluent limitations are given in the Code of Federal Regulations (CFR) 40 CFR Part 133 (federal) and in Chapter 173-221 WAC (state). These regulations are performance standards that constitute all known available and reasonable methods of prevention, control, and treatment for municipal wastewater.

The following technology-based limits for pH, fecal coliform, BOD_5, and TSS are taken from Chapter 173-221 WAC; the % removal requirements have been relaxed per WAC 173-221-050 (3) at the request of the Permittee due to treating CSO flows in the plant:

Table 5: Technology-based Limits.

Parameter	Limit
pH:	shall be within the range of 6 to 9 standard units.
Fecal Coliform Bacteria	Monthly Geometric Mean = 200 organisms/100 mL Weekly Geometric Mean = 400 organisms/100 mL
BOD_5 (concentration)	Average Monthly Limit is the most stringent of the following: - 30 mg/L - may not exceed twenty percent (20%) of the average influent concentration Average Weekly Limit = 45 mg/L
TSS (concentration)	Average Monthly Limit is the most stringent of the following: - 30 mg/L - may not exceed twenty percent (20%) of the average influent concentration Average Weekly Limit = 45 mg/L

The following technology-based mass limits for BOD_5 and TSS are based on WAC 173-220-130(3)(b) and 173-221-030(11)(b).

Monthly effluent mass loadings (lbs./day) were calculated as the maximum monthly design flow (5.6 MGD) x Concentration limit (30 mg/L) x 8.34 (conversion factor) = 1401 lbs./day.

The weekly average effluent mass loading is calculated as 1.5 x monthly loading = 2102 lbs./day.

The percent removal requirement for BOD_5 and TSS was relaxed because the WWTP processes CSO flows, that is sewage diluted by rainwater. Relaxing the removal percentage to 80% is based on BPJ. The influent strength regularly drops to about 150 mg/L. 15% of 150 mg/L is 22.5 mg/L; 20% of 150 mg/L is 30 mg/L. Removing 80% of the low strength influent during CSO episodes corresponds to the technology-based limits for effluent BOD_5 and TSS.

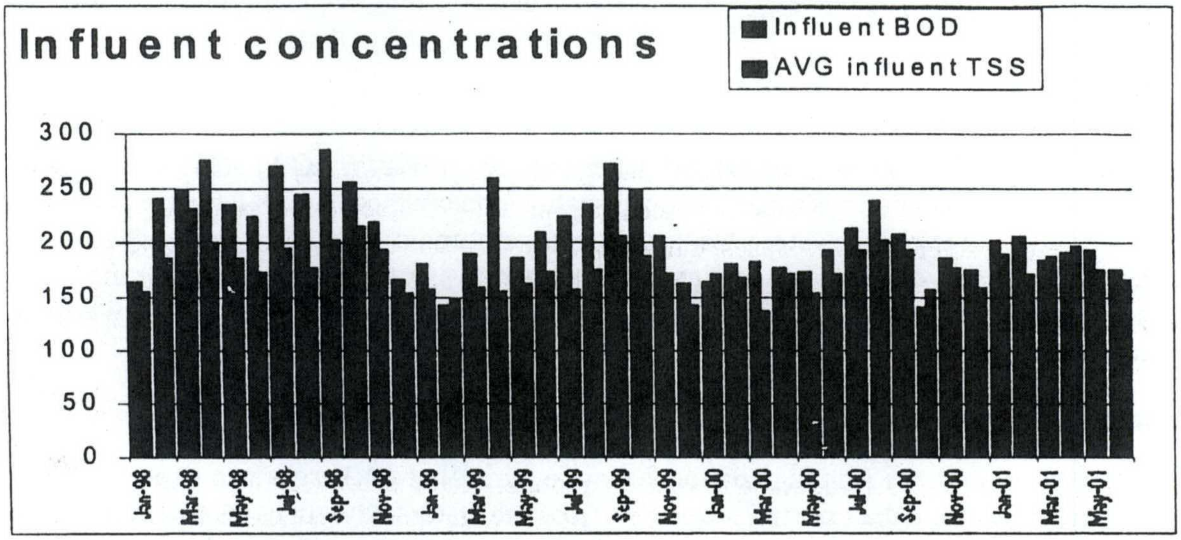

Figure 4: Influent TSS and BOD concentrations since CSO transport to the WWTP was initiated in 1998.

SURFACE WATER QUALITY-BASED EFFLUENT LIMITATIONS

In order to protect existing water quality and preserve the designated beneficial uses of Washington's surface waters, WAC 173-201A-060 states that waste discharge permits shall be conditioned such that the discharge will meet established Surface Water Quality Standards. The Washington State Surface Water Quality Standards (Chapter 173-201A WAC) is a state regulation designed to protect the beneficial uses of the surface waters of the state. Water quality-based effluent limitations may be based on an individual waste load allocation (WLA) or on a WLA developed during a basin-wide total maximum daily loading study (TMDL).

NUMERICAL CRITERIA FOR THE PROTECTION OF AQUATIC LIFE

"Numerical" water quality criteria are numerical values set forth in the State of Washington's Water Quality Standards for Surface Waters (Chapter 173-201A WAC). They specify the levels of pollutants allowed in a receiving water while remaining protective of aquatic life. Numerical criteria set forth in the Water Quality Standards are used along with chemical and physical data for the wastewater and receiving water to derive the effluent limits in the discharge permit. When surface water quality-based limits are more stringent or potentially more stringent than technology-based limitations, they must be used in a permit.

NUMERICAL CRITERIA FOR THE PROTECTION OF HUMAN HEALTH

The state was issued 91 numeric water quality criteria for the protection of human health by the U.S. EPA (EPA, 1992). These criteria are designed to protect humans from cancer and other diseases and are primarily applicable to fish and shellfish consumption and drinking water from surface waters.

NARRATIVE CRITERIA

In addition to numerical criteria, "narrative" water quality criteria (WAC 173-201A-030) limit toxic, radioactive, or deleterious material concentrations below those which have the potential to adversely affect characteristic water uses, cause acute or chronic toxicity to biota, impair aesthetic values, or adversely affect human health. Narrative criteria protect the specific beneficial uses of all fresh (WAC 173-201A-130) and marine (WAC 173-201A-140) waters in the State of Washington.

ANTIDEGRADATION

The State of Washington's Antidegradation Policy requires that discharges into a receiving water shall not further degrade the existing water quality of the water body. In cases where the natural conditions of a receiving water are of lower quality than the criteria assigned, the natural conditions shall constitute the water quality criteria. Similarly, when the natural conditions of a receiving water are of higher quality than the criteria assigned, the natural conditions shall constitute the water quality criteria. More information on the State Antidegradation Policy can be obtained by referring to WAC 173-201A-070.

The Department has reviewed existing records and is unable to determine if ambient water quality is either higher or lower than the designated classification criteria given in Chapter 173-201A WAC; therefore, the Department will use the designated classification criteria for this water body in the proposed permit. The discharges authorized by this proposed permit will not cause a loss of beneficial uses.

CRITICAL CONDITIONS

Surface water quality-based limits are derived for the water body's critical condition, which represents the receiving water and waste discharge condition with the highest potential for adverse impact on the aquatic biota, human health, and existing or characteristic water body uses.

The following technology-based mass limits for BOD_5 and TSS are based on WAC 173-220-130(3)(b) and 173-221-030(11)(b).

Monthly effluent mass loadings (lbs./day) were calculated as the maximum monthly design flow (5.6 MGD) x Concentration limit (30 mg/L) x 8.34 (conversion factor) = 1401 lbs./day.

The weekly average effluent mass loading is calculated as 1.5 x monthly loading = 2102 lbs./day.

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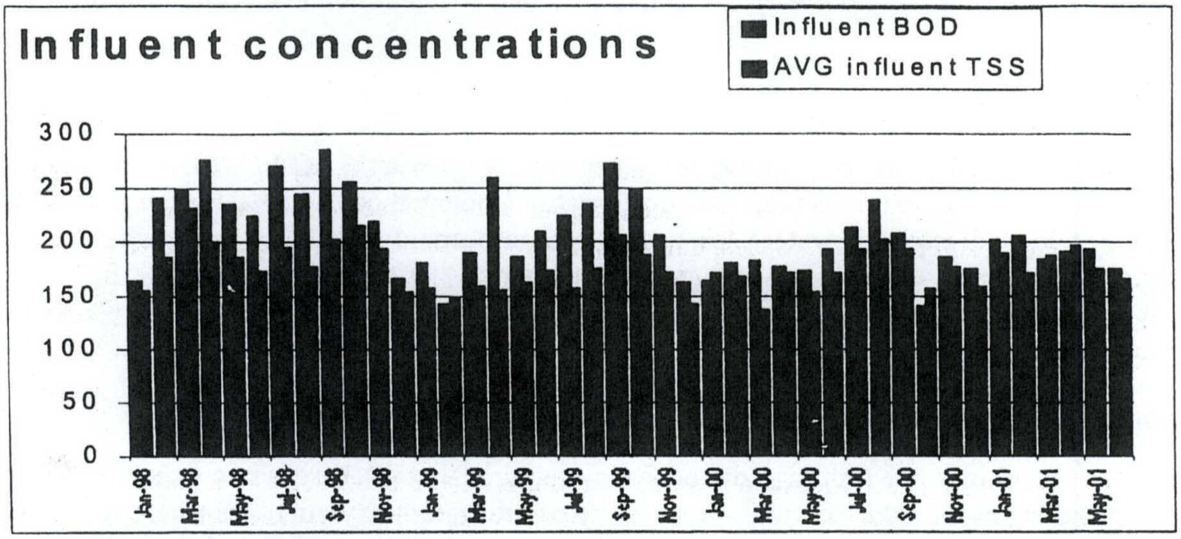

Figure 4: Influent TSS and BOD concentrations since CSO transport to the WWTP was initiated in 1998.

SURFACE WATER QUALITY-BASED EFFLUENT LIMITATIONS

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Mixing Zones

The Water Quality Standards allow the Department of Ecology to authorize mixing zones around a point of discharge in establishing surface water quality-based effluent limits. Both "acute" and "chronic" mixing zones may be authorized for pollutants that can have a toxic effect on the aquatic environment near the point of discharge. The concentration of pollutants at the boundary of these mixing zones may not exceed the numerical criteria for that type of zone. Mixing zones can only be authorized for discharges that are receiving all known, available, and reasonable methods of prevention, control, and treatment (AKART) and in accordance with other mixing zone requirements of WAC 173-201A-100.

The National Toxics Rule (EPA, 1992) allows the chronic mixing zone to be used to meet human health criteria.

Description of the Receiving Water

The facility discharges treated wastewater to the Skagit River, which is designated as a Class A receiving water in the vicinity of the treatment plant outfall. CSO outfalls are located within one mile upstream of the treatment plant outfall. Characteristic uses include the following: water supply (domestic, industrial, agricultural); stock watering; fish migration; fish rearing, spawning and harvesting; wildlife habitat; primary contact recreation; sport fishing; boating and aesthetic enjoyment; commerce and navigation.

Water quality of this class shall markedly and uniformly exceed the requirements for all or substantially all uses.

Surface Water Quality Criteria

Applicable criteria are defined in Chapter 173-201A WAC for aquatic biota. In addition, U.S. EPA has promulgated human health criteria for toxic pollutants (EPA, 1992). Criteria for this discharge are summarized below:

Fecal Coliforms	100 organisms/100 mL maximum geometric mean
Dissolved Oxygen	8 mg/L minimum
Temperature	18 degrees Celsius maximum or incremental increases above background
pH	6.5 to 8.5 standard units
Turbidity	less than 5 NTUs above background
Toxics	No toxics in toxic amounts (see Appendix C for numeric criteria for toxics of concern for this discharge)

CONSIDERATION OF SURFACE WATER QUALITY-BASED LIMITS FOR NUMERIC CRITERIA

Pollutant concentrations in the proposed discharge exceed water quality criteria with technology-based controls which the Department has determined to be AKART. A mixing zone is authorized in accordance with the geometric configuration, flow restriction, and other restrictions for mixing zones in Chapter 173-201A WAC and is defined as follows:

The dilution factors of effluent to receiving water that occur within these zones have been determined at the critical condition by the use of method of Fischer et al. – the Department's Rivplume model spreadsheet. The City of Mount Vernon submitted a study of the mixing characteristics at the wastewater treatment plant outfall, (Cosmopolitan, 2000) that included acute and chronic dilution factors for the current outfall. Based on the dye study and field measurements in that report, the Department calibrated the model and used the calibrated model to estimate acute and chronic dilution factors for the outfall (see Appendix C for the spreadsheet outputs). The mixing zone study concluded that the outfall must be improved to provide for compliance with State water quality standards as effluent flows increase. The estimated dilution associated with this discharge has reduced significantly in relation to the previous permit. The dilution factors have been determined to be:

Table 6: Estimated Dilution Factors for the WWTP Outfall into the Skagit River.

	Acute	Chronic
Aquatic Life	5:1	35:1
Human Health, Carcinogen	[redacted]	Use 35:1
Human Health, Non-carcinogen		Use 35:1

Pollutants in an effluent may affect the aquatic environment near the point of discharge (near-field) or at a considerable distance from the point of discharge (far-field). Toxic pollutants, for example, are near-field pollutants--their adverse effects diminish rapidly with mixing in the receiving water. Conversely, a pollutant such as BOD is a far-field pollutant whose adverse effect occurs away from the discharge even after dilution has occurred. Thus, the method of calculating water quality-based effluent limits varies with the point at which the pollutant has its maximum effect.

The derivation of water quality-based limits also takes into account the variability of the pollutant concentrations in both the effluent and the receiving water.

The Department analyzed the capacity of the lower Skagit River for pollutants discharged from point sources that have significant impact on dissolved oxygen levels in the water column. These pollutants are $CBOD_5$ or BOD_5 and ammonia (as NH_3-N). The results of the study are documented in three Department of Ecology documents: <u>Lower Skagit Total Maximum Daily Load Data Summary</u> (Ecology, 1996); <u>Lower Skagit Total Maximum Daily Load Water Quality Study</u> (Ecology, 1997); and <u>Lower Skagit Total Maximum Daily Load Submittal Report</u> (Ecology, 2000). Final waste load allocations (WLAs) and permit limitations for the City of Mount Vernon WWTP are listed in the latter publication and are shown in the following table.

Table 7: WLAs and Corresponding Permit Limits for the WWTP.

	Time in effect	$CBOD_5$ (lbs./day)	BOD_5 (lbs./day)	NH_3-N (lbs./day)
Alternative WLAs – for future use	*July 1 to Nov. 15*	*2712*	*3051*	*678*
WLAs	July 1 to Nov. 15	1902	2140	1188
Monthly average limit	July 1 to Oct. 31	1407	1583	922
Maximum daily (NH_3) or weekly (BOD_5 or $CBOD_5$)	July 1 to Nov. 15	1902	2140	1188

The alternative WLAs in the first row of the table demonstrate trading reduced ammonia output for increased BOD_5 output; this exchange provides for increased flows due to population growth offset by constructing new treatment facilities capable of treating for reduced ammonia levels. The permit contains permit limits based on waste load allocations with higher ammonia and lower BOD_5.

The permit shows limits for both BOD_5 or $CBOD_5$, the Permittee may measure either one to demonstrate compliance. The BOD_5 test, the measure used historically for the plant, also measures the influence of ammonia on oxygen uptake to a limited extent. The $CBOD_5$ test suppresses the influence of ammonia on oxygen uptake in the laboratory. BOD_5 testing is required for other reasons, $CBOD_5$ testing may be employed to more precisely demonstrate compliance with the WLAs derived in the Skagit TMDL.

Temperature--The impact of the discharge on the temperature of the receiving water was modeled by simple mixing analysis at critical condition. The receiving water temperature at the critical condition is 15.2° C and the effluent temperature is 20° C. The predicted resultant temperature at the boundary of the chronic mixing zone is 15.3° C and the incremental rise is 0.1° C.

Under critical conditions there is no predicted violation of the temperature standard; no effluent limitation for temperature was placed in the permit.

pH--The impact of pH and temperature were modeled using the calculations from EPA, 1988 (see Appendix C). The input variables were dilution factor 35:1, upstream temperature 15.2° C, upstream pH 7.7, upstream alkalinity 20 (as mg $CaCO_3$/L), effluent temperature 20° C, a range of effluent pH values, and effluent alkalinity 195 (as mg $CaCO_3$/L).

Under critical conditions there was a prediction of a violation of the pH criteria for the receiving water. An effluent limit of 6.6 to 9.0 for pH was found to meet the water quality criterion for pH. These limits are included in the permit with a compliance schedule that requires plans for meeting the limits to be submitted by June 30, 2003. The plant lacks the equipment to meet this new limitation at this time. An interim limit of 6.0 to 9.0, the technology-based limit is used as an interim limit.

Fecal Coliform--The lower Skagit River is listed on the State's 303d list (waters that fail to meet standards) for fecal coliform bacteria. The Lower Skagit Total Maximum Daily Load Water Quality Study (Ecology, 1997) concluded that compliance with the technology-based limit for fecal coliform was an appropriate waste load allocation for bacteria at the WWTP outfall. Reduction of CSO events at the Mount Vernon CSO outfalls to once per year is a sufficient reduction in bacteria output to return the lower Skagit to compliance with the bacteria standard (coupled with reductions in other sources outside of the City of Mount Vernon).

Toxic Pollutants--Federal regulations (40 CFR 122.44) require NPDES permits to contain effluent limits for toxic chemicals in an effluent whenever there is a reasonable potential for those chemicals to exceed the surface water quality criteria. This process occurs concurrently with the derivation of technology-based effluent limits. Facilities with technology-based effluent limits defined in regulation are not exempted from meeting the Water Quality Standards for Surface Waters or from having surface water quality-based effluent limits.

The following toxics were detected in the discharge: ammonia, copper, zinc, BHC-gamma (lindane), bis(2-ethylhexyl) phthalate, chloroform, and cyanide. The Department conducted an analysis with procedures given in EPA, 1991, to determine if there was a reasonable potential for any of these pollutants to exceed water quality standards outside of the mixing zones during critical conditions (see Appendix C). Effluent limitations are required for any pollutants with potential to exceed standards. The critical condition in this case occurs during the fall. The parameters used in the critical condition modeling are as follows: acute dilution factor 5:1, chronic dilution factor 35:1, receiving water temperature 15.4° C, pH of 7.8, and receiving water hardness 25 (as mg $CaCO_3$/L).

Water quality criteria for metals in Chapter 173-201A WAC are based on the dissolved fraction of the metal.

Effluent limits were derived for copper, zinc, ammonia, and chlorine, which were determined to have a reasonable potential to cause a violation of the Water Quality Standards. Chlorine is used to disinfect the effluent, then is chemically neutralized. Effluent limits were calculated using methods from EPA, 1991, as shown in Appendix C. The ammonia-N water quality criteria are taken from the mixing zone study (Cosmopolitan, 2000). That report derived the ammonia criteria based on ambient conditions during the summer and fall months from 1992 to 1997. Ammonia limits are based on a normal distribution instead of the lognormal distribution frequently used to model discharges because the data follows the normal distribution. The resultant effluent limits are as follows [8.34 x 5.6 MGD x concentration limit (mg/L) = mass limit]:

Table 8: Water Quality-Based Limits to Meet Numerical Water Quality Standards.

Parameter	Average Monthly Limit	Maximum Daily Limit
NH_3-N (as N)	30 mg/L, 1400 lbs./day	41 mg/L
copper	9.4 ug/L, 0.44 lbs./day	16.6 ug/L
zinc	88.4 ug/L, 4.13 lbs./day	177.4 ug/L
chlorine	47.4 ug/L, 2.21 lbs./day	95.0 ug/L

The permit contains a compliance schedule for meeting the water quality-based limits for copper. The WWTP has collected extensive data on copper levels in the effluent (see Appendix C). The WWTP can not meet the copper limit based on the data collected from 1993 to 1995; the mean copper level in the effluent during that period was 18 ug/L. The City has submitted an analysis of effluent limitations for several metals and dilution available by modifying the outfall. That report concluded that modifying the outfall is necessary to meet water quality standards for several parameters as WWTP flows increase. The City has no industrial dischargers that discharge metals. The Department also will require that the City submit a sampling plan to assure that reported values of trace metals are as accurate as possible. An interim copper limitation based on previous sampling by the facility is included in the permit to assure the level of copper discharged does not increase.

The interim limit was calculated with the spreadsheet used for other permit limitations, but the 98% maximum of 35 ug/L was entered as a WLA (maximum amount allowed to be discharged, then limits were calculated from that WLA). The calculation yielded limits of 21 ug/L monthly average and 35 ug/L daily maximum. Note that copper is present in multivitamin supplements at a level of 1,000 ug per tablet and copper pipe is commonly used in household plumbing.

The Permittee may provide data clearly demonstrating the seasonal partitioning of the dissolved metal in the ambient water in relation to an effluent discharge. Water quality criteria for metals in Chapter 173-201A WAC are based on the dissolved fraction of the metal. Metals criteria may be adjusted on a site-specific basis when data is available clearly demonstrating the seasonal partitioning in the ambient water in relation to an effluent discharge.

Metals criteria may also be adjusted using the water effects ratio approach established by USEPA, as generally guided by the procedures in USEPA Water Quality Standards Handbook, December 1983, as supplemented or replaced.

WHOLE EFFLUENT TOXICITY

The Water Quality Standards for Surface Waters require that the effluent not cause toxic effects in the receiving waters. Many toxic pollutants cannot be detected by commonly available detection methods. However, toxicity can be measured directly by exposing living organisms to the wastewater in laboratory tests and measuring the response of the organisms. Toxicity tests measure the aggregate toxicity of the whole effluent, and therefore this approach is called whole effluent toxicity (WET) testing. Some WET tests measure acute toxicity (death is the endpoint) and other WET tests measure chronic toxicity (growth, reproduction, and mortality are measured as endpoints).

The Department sets performance standards and effluent limits (shown in Table 9) in Chapter 173-205 WAC. A Permittee that meets the performance standards during effluent characterization or during routine testing required in a permit does not receive permit limits. If a Permittee fails to meet a performance standard, they receive limits for WET and perform routine testing during a permit cycle. Acute WET and chronic WET are treated as two separate limits, so a Permittee can receive a limit for acute WET, chronic WET, or both. When the Permittee meets the performance standard for a period of three (3) years during the routine testing, then the effluent limit can be removed from the permit.

Table 9: Performance Criteria and Limits for WET.
NOEC means no observable effects concentration.

Test description	Performance criteria in state regulation	Permit limit
96-hour fathead minnow acute toxicity	Median of 80% and minimum of 65% survival required in 100% effluent.	Median of 80% and minimum of 65% survival required in 20% effluent.
48-hour daphnid acute toxicity test	Median of 80% and minimum of 65% survival required in 100% effluent.	Median of 80% and minimum of 65% survival required in 20% effluent.
7-day fathead minnow survival and growth test	NOEC equal to or greater than 20% effluent	NOEC equal to or greater than 3% effluent
Chronic (>7-days) Ceriodaphnia survival and reproduction test	NOEC equal to or greater than 20% effluent	NOEC equal to or greater than 3% effluent

Acute toxicity tests measure mortality as the significant response to the toxicity of the effluent. Dischargers who monitor their wastewater with acute toxicity tests are providing an indication of the potential lethal effect of the effluent to organisms in the receiving environment.

Chronic toxicity tests measure various sublethal toxic responses such as retarded growth or reduced reproduction. Chronic toxicity tests often involve either a complete life cycle test of an organism with an extremely short life cycle or a partial life cycle test on a critical stage of one of a test organism's life cycles. Organism survival is also measured in some chronic toxicity tests.

Accredited WET testing laboratories have the proper WET testing protocols, data requirements, and reporting format. Accredited laboratories are knowledgeable about WET testing and capable of calculating an NOEC, LC_{50}, EC_{50}, IC_{25}, etc. All accredited labs have been provided the most recent version of the Department of Ecology Publication # WQ-R-95-80, *Laboratory Guidance and Whole Effluent Toxicity Test Review Criteria,* which is referenced in the permit. Any Permittee interested in receiving a copy of this publication may call the Ecology Publications Distribution Center (360-407-7472) for a copy. Ecology recommends that Permittees send a copy of the acute or chronic toxicity sections(s) of their permits to their laboratory of choice.

The Permittee's effluent has been determined to have the potential to contain toxic chemicals. Effluent tests for acute and chronic toxicity were conducted during the previous permit term. Tables 10 & 11 show the test results. These tests are dated and were not necessarily conducted in accordance with state regulation (The regulation covering WET testing was promulgated after the 1993 permit was issued.). In accordance with WAC 173-205, the Permittee conducts a WET effluent characterization for the following reasons:

- Toxicity was detected during previous WET tests.
- Average plant flows have increased since the previous tests were conducted. The acute and chronic dilution factors have been revised significantly based on new information. A dechlorination system was added to the treatment system in 1998.
- Some testing was done with species that are not used in the current regulation and the test data is old.

Table 10: Mt. Vernon WWTP Acute WET Test Results as % Survival in 100% Effluent.

Lab	Test	Species	Sample Date	Test Date	Protocol	Duration	% Survival
WANCA	KJOH380	Rainbow Trout	7/26/1993	7/27/1993	EPAA 91	96 hours	90
WAPTL	KJOH383	Fathead Minnow	9/20/1993	9/21/1993	EPAF 89	96 hours	2.5
WANCA	KJOH382	Rainbow Trout	10/25/1993	10/26/1993	EPAA 91	96 hours	75
WAPTL	KJOH385	Fathead Minnow	4/25/1994	4/26/1994	EPAF 89	96 hours	0
WAPTL	KJOH387	Fathead Minnow	6/14/1994	6/15/1994	EPAF 89	96 hours	0
WAPTL	AQTX1558	*Daphnia pulex*	11/4/1997	11/4/1997	EPAA 91	48 hours	95
WAPTL	AQTX1818	Fathead Minnow	5/19/1998	5/19/1998	EPAA 91	96 hours	97.5

Table 11: Mt. Vernon WWTP Chronic WET Test Results as NOEC/LOEC in % Effluent.

Lab	Test	Species	Sample Date	Test Date	Protocol	End Point	NOEC	LOEC
WAPTL	KJOH381	*Ceriodaphnia dubia*	8/23/1993	8/24/1993	EPAF 89	7d Proportion Survived	50	100
						Reproduction	25	50
WAPTL	KJOH383	Fathead Minnow	9/20/1993	9/21/1993	EPAF 89	7d Proportion Survived	25	50
						Mean Weight	50	100
						Mean Biomass	25	50
WAPTL	KJOH384	*Ceriodaphnia dubia*	3/14/1994	3/15/1994	EPAF 89	7d Proportion Survived	50	100
						Reproduction	25	50
WAPTL	KJOH385	Fathead Minnow	4/25/1994	4/26/1994	EPAF 89	7d Proportion Survived	50	100
						Mean Weight	25	50
						Mean Biomass	25	50
WAPTL	KJOH386	*Ceriodaphnia dubia*	5/31/1994	6/1/1994	EPAF 89	7d Proportion Survived	50	100
						Reproduction	50	100
WAPTL	KJOH387	Fathead Minnow	6/14/1994	6/15/1994	EPAF 89	7d Proportion Survived	25	50
						Mean Weight	12.5	25
						Mean Biomass	6.25	12.5

Lab	Test	Species	Sample Date	Test Date	Protocol	End Point	NOEC	LOEC
WAAVO	AQTX1561	Fathead Minnow	10/14/1997	10/14/1997	EPAF 94	7d Proportion Survived	10	100
						Mean Weight	100	> 100
						Mean Biomass	10	100
WAAVO	AQTX1560	*Ceriodaphnia dubia*	10/21/1997	10/21/1997	EPAF 94	7d Proportion Survived	10	100
						Reproduction	5	10

If the WET performance criteria in Table 9 are met during the one-year WET characterization, then no additional tests are required. If the Permittee fails to meet the performance criteria during the characterization testing, then routine WET testing is required to show compliance with effluent limits. If the permit limits are not met, then the Permittee is required to increase testing frequency, locate the source of the toxicity, and eliminate it. Additional characterization tests may be required for the next permit application.

The acute toxicity limit is set relative to the zone of acute criteria exceedance (acute mixing zone) established in accordance with WAC 173-201A-100. The acute critical effluent concentration (ACEC) is the concentration of effluent existing at the boundary of the acute mixing zone during critical conditions. This value is 20% (1/5) effluent. The acute toxicity limit is no statistically significant difference in test organism survival between the ACEC, 20% of the effluent, and the control.

The chronic toxicity limit is set relative to the mixing zone established in accordance with WAC 173-201A-100. The chronic critical effluent concentration (CCEC) is the concentration of effluent existing at the boundary of the mixing zone during critical conditions. This value is 3% (1/35) effluent. Monitoring for compliance with a chronic toxicity limit is accomplished by conducting a chronic toxicity test using a sample of effluent diluted to equal the CCEC and comparing test organism response in the CCEC to organism response in nontoxic control water. The Permittee is in compliance with the chronic toxicity limit if there is no statistically significant difference in test organism response between the CCEC of 3% effluent and the control.

If the Permittee makes process or material changes which, in the Department's opinion, results in an increased potential for effluent toxicity, then the Department may require additional effluent characterization in a regulatory order, by permit modification, or in the permit renewal. Toxicity is assumed to have increased if WET testing conducted for submission with a permit application fails to meet the performance standards in WAC 173-205-020, "whole effluent toxicity performance standard." The Permittee may demonstrate to the Department that changes have not increased effluent toxicity by performing additional WET testing after the time the process or material changes have been made.

HUMAN HEALTH

Washington's water quality standards now include 91 numeric health-based criteria that must be considered in NPDES permits. These criteria were promulgated for the state by the U.S. EPA in its National Toxics Rule (Federal Register, Volume 57, No. 246, Tuesday, December 22, 1992).

The Department has determined that the effluent is likely to have chemicals of concern for human health. The discharger's high priority status is based on the discharger's status as a major discharger.

A determination of the discharge's potential to cause an exceedance of the water quality standards was conducted as required by 40 CFR 122.44(d). The reasonable potential determination was evaluated with procedures given in the Technical Support Document for Water Quality-Based Toxics Control (EPA/505/2-90-001) and the Department's Permit Writer's Manual (Ecology Publication 92-109, July 1994). The determination indicated that the discharge has no reasonable potential to cause a violation of water quality standards, thus an effluent limit is not warranted.

SEDIMENT QUALITY

The Department has been unable to determine at this time the potential for this discharge to cause a violation of sediment quality standards. The Department has not set specific chemical criteria for freshwater sediments. The treatment system employed is designed to remove significant portions of solids, so sediment deposition from this discharge is unlikely. If the Department determines in the future that there is a potential for violation of the Sediment Quality Standards, an order will be issued to require the Permittee to demonstrate that either the point of discharge is not an area of deposition or, if the point of discharge is a depositional area, that there is not an accumulation of toxics in the sediments.

COMPARISON OF EFFLUENT LIMITS WITH THE EXISTING PERMIT ISSUED in 1993

Parameter	Limits in 1993 permit	Proposed technology-based Limits	Proposed WQ toxicity Limits	Proposed TMDL Limits
BOD_5 (November through June)	monthly average 30 mg/L, 1000 lbs./day weekly maximum 45 mg/L, 1500 lbs./day	**monthly average** **30 mg/L, 1401 lbs./day** **weekly maximum** **45 mg/L, 2102 lbs./day**	Not applicable	Not applicable
BOD_5 (July through October)	monthly average 30 mg/L, 1000 lbs./day weekly maximum 45 mg/L, 1500 lbs./day	**monthly average** **30 mg/L, 1401 lbs./day** **weekly maximum** **45 mg/L, 2102 lbs./day**	Not applicable	monthly average 1583 lbs./day weekly maximum 2140 lbs./day
Ammonia as NH_3-N (November through June)	Not applicable	Not applicable	**monthly average** **30 mg/L, 1400 lbs./day** **daily maximum** **41 mg/L**	Not applicable
Ammonia as NH_3-N (July through October)	Not applicable	Not applicable	**monthly average** **30 mg/L,** 1400 lbs./day **daily maximum** **41 mg/L**	**monthly average** **922 lbs./day** **daily maximum** **1188 lbs./day**
TSS	monthly average 30 mg/L, 1000 lbs./day weekly maximum 45 mg/L, 1500 lbs./day	monthly average 30 mg/L, 1401 lbs./day weekly maximum 45 mg/L, 2102 lbs./day	Not applicable	Not applicable
pH	shall be within the range of 6 to 9 standard units	shall be within the range of 6.0 to 9 standard units	**shall be within the range of 6.6 to 9 standard units**	Not applicable

Parameter	Limits in 1993 permit	Proposed technology-based Limits	Proposed WQ toxicity Limits	Proposed TMDL Limits
Fecal Coliform Bacteria	monthly average 200/100 mL weekly maximum 400/100 mL	**monthly average 200/100 mL weekly maximum 400/100 mL**	Not applicable	monthly average 200/100 mL weekly maximum 400/100 mL
Total Residual Chlorine	(Interim) 0.5 mg/L monthly average 0.75 mg/L weekly maximum	Not applicable	**monthly average 0.05 mg/L, 2.2 lbs./day daily maximum 0.10 mg/L**	Not applicable
Copper (interim)	none	Not applicable	**monthly average 21 ug/L, 0.98 lbs./day daily maximum 35 ug/L**	Not applicable
Copper	monthly average 28 ug/L daily maximum 48 ug/L	Not applicable	**monthly average 9.4 ug/L, 0.44 lbs./day daily maximum 16.6 ug/L**	Not applicable
Zinc	none	Not applicable	**monthly average 88.4 ug/L, 4.13 lbs./day daily maximum 177.4 ug/L**	Not applicable

(Note: The copper limit in the 1993 permit was removed by modification.)

MONITORING REQUIREMENTS

Monitoring, recording, and reporting are required (WAC 173-220-210 and 40 CFR 122.41) to verify that the treatment process is functioning correctly and the effluent limitations are being achieved.

Monitoring of sludge quantity and quality is necessary to determine the appropriate uses of the sludge. Sludge monitoring is required by the current state and local solid waste management program and also by EPA under 40 CFR 503.

The monitoring schedule is detailed in the proposed permit under Condition S.2. Specified monitoring frequencies take into account the quantity and variability of discharge, the treatment method, past compliance, significance of pollutants, and cost of monitoring. The required monitoring frequency is consistent with agency guidance given in the current version of Ecology's *Permit Writer's Manual* (July 1994) for an activated sludge plant with annual average design flow 2.0 to 5.0 MGD.

WET testing frequencies are also based on Ecology's *Permit Writer's Manual*. The discharge meets a Rank 4 that calls for four acute and two chronic WET tests per year. The effluent contains low levels of toxic pollutants listed in Appendix D of 40 CFR Part 122 (15 points), has had toxicity detected in past tests (10 points), has an average annual flow between 0.5 and 12.5 MGD (10 points), and has a CCEC between 2% and 4% effluent (10 points). This yields a score of (15+10) x (10+10) = 500; Rank 4 runs from 100 to 750 points in the *Permit Writer's Manual.*

LAB ACCREDITATION

With the exception of certain parameters, the permit requires all monitoring data to be prepared by a laboratory registered or accredited under the provisions of Chapter 173-50 WAC, *Accreditation of Environmental Laboratories.* The laboratory at this facility is accredited for fecal coliform, ammonia, BOD_5, TSS, Total Residual Chlorine, and pH.

OTHER PERMIT CONDITIONS

REPORTING AND RECORDKEEPING

The conditions of S3. are based on the authority to specify any appropriate reporting and record keeping requirements to prevent and control waste discharges (WAC 273-220-210).

PREVENTION OF FACILITY OVERLOADING

Overloading of the treatment plant is a violation of the terms and conditions of the permit. To prevent this from occurring, RCW 90.48.110 and WAC 173-220-150 require the Permittee to take the actions detailed in proposed permit requirement S.4. to plan expansions or modifications before existing capacity is reached and to report and correct conditions that could result in new or increased discharges of pollutants.

OPERATION AND MAINTENANCE (O&M)

The proposed permit contains condition S.5. as authorized under RCW 90.48.110, WAC 173-220-150, Chapter 173-230 WAC, and WAC 173-240-080. It is included to ensure proper operation and regular maintenance of equipment, and to ensure that adequate safeguards are taken so that constructed facilities are used to their optimum potential in terms of pollutant capture and treatment.

RESIDUAL SOLIDS HANDLING

To prevent water quality problems, the Permittee is required in permit condition S7. to store and handle all residual solids (grit, screenings, scum, sludge, and other solid waste) in accordance with the requirements of RCW 90.48.080 and State Water Quality Standards.

The final use and disposal of sewage sludge from this facility is regulated by U.S. EPA under 40 CFR 503. The disposal of other solid waste is under the jurisdiction of the local health department where the material is disposed of.

PRETREATMENT

The Department administers the pretreatment program for this WWTP. Under this delegation of authority, the Department writes and issues wastewater discharge permits for significant industrial users (SIUs) discharging to the City of Mount Vernon sewer system. The City may refuse to accept industrial discharges to their system. The Permittee may request delegation from the Department to manage their own pretreatment program at any time.

Currently two industrial users discharge to the Mount Vernon WWTP. Draper Valley Farms processes chickens and discharges wash water to the system. Hallmark Refining reclaims photo finishing liquids and discharges trace levels of metals to the system.

Federal requirements for a Pretreatment Program are contained in Title 40, part 403, of the Code of Federal Regulations. State requirements for the discharge of nondomestic wastewater to a public sewer system are set Chapter 173-216 WAC. The Department requires SIUs and other industrial dischargers to obtain a State Waste Discharge Permit prior to discharging to the sewer system. The State regulation exempts wastewater discharges that are similar in character to domestic wastewater from the permit requirement. The Department has the final authority to determine whether a permit is required of an industrial or commercial discharger. Any industrial or commercial facility that discharges wastewater generated should contact the Department to determine if a permit is required. Industrial dischargers are required to apply for a discharge permit sixty (60) days prior to commencing discharge (WAC 173-216-110). The permit requires that the City notify the Department of any new or previously undetected industrial discharges immediately to assure compliance with these requirements.

The permit requires the Permittee to provide a list of industrial users with the next permit application. Methods for generating the list may include review of business tax licenses in the area, water billing records, local telephone directories, and connection authorization records. The City of Mount Vernon should require that the various city departments involved with issuing building permits or performing inspections of commercial and industrial facilities be aware of the requirements for obtaining a state permit for wastewater discharges. The pretreatment program protects the treatment plant from breakdowns caused by toxic discharges to the sewer system.

The permit prohibits the POTW from authorizing or permitting an industrial discharger to discharge certain types of waste into the sanitary sewer. These prohibitions are taken directly from 40 CFR Part 403 and WAC 173-216-060. The prohibitions are included to prevent pass through or interference, upset of the plant processes, damage to the collection or treatment system, hazardous conditions for plant personnel and the public. Discharge of water that does not need treatment in a POTW is prohibited except under exceptional circumstances.

COMBINED SEWER OVERFLOWS (CSOs)

In 1996 the City of Mount Vernon entered into Order on Consent DE 96WQ-N105 in which the City agreed to a schedule to reduce CSOs to once per year by the year 2015. The City has remained in compliance with the order. The City completed a comprehensive sewer plan including long term plans for correcting overflows. They designed and constructed the Central CSO Interceptor to store CSO flows for treatment at the WWTP in 1998. That project also installed flow measure and sampling equipment on the CSO control structure to provide information for measuring and characterizing CSO episodes. Mount Vernon submits an annual report to summarize CSO discharge quantities and reduction related accomplishments to the Department. Data from annual CSO reports is summarized in Table 12.

Table 12: Summary of CSO Discharge from 1996 to 2000.

	Total Annual Flow (MG)	**Total Number of Events**	**Annual Precipitation**	**Comments**
Baseline[1]	116	130	32.0	Estimated average before CSO controls in place, CSO episodes are precipitation dependent.
1996	124	Unknown	34.4	No direct data flow or event count available.
1997	119	About 89	33.7	Event Count Based on Division Street
1998	17	4	33.4	CSO controls went online in March 1998
1999	11	7	37.2	Operational controls of CSO Regulator in tandem with WWTP refined. Sampling provided data on overflow characteristics.
2000	12	3	24	Sampling provided data on overflow characteristics.

[1] Baseline is from Comprehensive Sewer and Combined Sewer Overflow Reduction Plans for City of Mount Vernon, R.W. Beck, 1991.

The impact of the Mount Vernon CSOs on water quality in the Skagit River has been significantly reduced by the improvements constructed over the last five years. Sampling of CSOs in 1999 averaged a BOD_5 concentration of 24 mg/L and TSS concentration of 50 mg/L based on composite samples from the overflows. City staff sampled fecal coliform bacteria immediately upstream of the CSO discharges at the Division Street Bridge and then downstream of the CSO outfalls and immediately upstream of the WWTP outfall. The differences between the two measurements ranged from 150 to 1400 cfu per 100 mL. The downstream values ranged from 230 to 1483 cfu per 100 mL. The BOD_5 concentration of the CSO generally meets the secondary treatment standard of 30 mg/L monthly average, 45 mg/L weekly maximum. The TSS concentration sometimes exceeds the standard of 30 mg/L monthly average, 45 mg/L weekly maximum. The fecal coliform bacteria concentration in the river is elevated during CSO events, but the reduction in volume and duration of the events provide significant improvement in water quality.

Other measures cited by the City in reducing the quantity and impact of CSOs on the Skagit River are installation of separate storm and sanitary pipes in conjunction with major street improvement projects, household and small business hazardous waste collection, street sweeping, and new equipment purchases to enhance the function and reliability of the CSO transport system.

The text of ORDER ON CONSENT No. DE 96WQ-N105 is included in Appendix D. Remaining milestones in the Order are:

- Upgrade the Park Street Pump Station with new pumps (Improvement number CA-1a) no later than January 1, 2015.
- Upgrade wastewater treatment plant to provide treatment/storage of CSO flows to reduce overflow events to an average of one per year no later than January 1, 2015.

In accordance with RCW 90.48.480 and Chapter 173-245 WAC, proposed permit Condition S10 requires the Permittee to submit an annual Combined Sewer Overflow (CSO) report and to update its CSO reduction plan at the time of permit renewal. Condition S10 also includes requirements for sampling CSO discharges at least once per year, although sampling for bacteria levels in the Skagit River is required only if the samples can be collected safely. The permit also requires the Permittee to maintain notification signs at the CSO outfalls.

GENERAL CONDITIONS

General Conditions are based directly on state and federal law and regulations and have been standardized for all individual municipal NPDES permits issued by the Department.

Condition G1 requires responsible officials or their designated representatives to sign submittals to the Department. Condition G2 requires the Permittee to allow the Department to access the treatment system, production facility, and records related to the permit. Condition G3 specifies conditions for modifying, suspending, or terminating the permit. Condition G4 requires the Permittee to apply to the Department prior to increasing or varying the discharge from the levels stated in the permit application. Condition G5 requires the Permittee to construct, modify, and operate the permitted facility in accordance with approved engineering documents. Condition G6 prohibits the Permittee from using the permit as a basis for violating any laws, statutes, or regulations. Conditions G7 relates to permit renewal. Condition G8 prohibits the reintroduction of removed substances back into the effluent. Condition G9 states that the Department will modify or revoke and reissue the permit to conform to more stringent toxic effluent standards or prohibitions. Condition G10 incorporates by reference all other requirements of 40 CFR 122.41 and 122.42. Condition G11 notifies the Permittee that additional monitoring requirements may be established by the Department. Condition G12 requires the payment of permit fees. Condition G13 describes the penalties for violating permit conditions.

PERMIT ISSUANCE PROCEDURES

PERMIT MODIFICATIONS

The Department may modify this permit to impose numerical limitations, if necessary, to meet Water Quality Standards, Sediment Quality Standards, or Ground Water Standards, based on new information obtained from sources such as inspections, effluent monitoring, outfall studies, and effluent mixing studies. The Department may also modify the permit to reflect changes or improvements to the treatment facilities.

The Department may also modify this permit as a result of new or amended state or federal regulations.

RECOMMENDATION FOR PERMIT ISSUANCE

This proposed permit meets all statutory requirements for authorizing a wastewater discharge, including those limitations and conditions believed necessary to protect human health, aquatic life, and the beneficial uses of waters of the State of Washington.- The Department proposes that this permit be issued for two (2) years.

REFERENCES FOR TEXT AND APPENDICES

Cosmopolitan Engineering Group

2000. Mount Vernon WWTP Mixing Zone Study. Unpublished report to the Department of Ecology from the City of Mount Vernon.

Environmental Protection Agency (EPA)

1992. National Toxics Rule. Federal Register, V. 57, No. 246, Tuesday, December 22, 1992.

1991. Technical Support Document for Water Quality-based Toxics Control. EPA/505/2-90-001.

1988. Technical Guidance on Supplementary Stream Design Conditions for Steady State Modeling. USEPA Office of Water, Washington, D.C.

1985. Water Quality Assessment: A Screening Procedure for Toxic and Conventional Pollutants in Surface and Ground Water. EPA/600/6-85/002a.

1983. Water Quality Standards Handbook. USEPA Office of Water, Washington, D.C.

Fischer, et al.

1979. Mixing in Inland and Coastal Waters. New York: Academic Press.

Metcalf and Eddy.

1991. Wastewater Engineering, Treatment, Disposal, and Reuse. Third Edition.

R.W. Beck.

1995. Mount Vernon Wastewater Treatment Plant Evaluation. Unpublished engineering report submitted to the Department of Ecology by the City of Mount Vernon.

1991. Comprehensive Sewer and Combined Sewer Overflow Reduction Plans for City of Mount Vernon. Unpublished technical report submitted to the Department of Ecology by the City of Mount Vernon.

Washington State Department of Ecology.

1994. Permit Writer's Manual. Publication Number 92-109

2000. Lower Skagit River Dissolved Oxygen Total Maximum Daily Load Submittal Report. Publication Number 00-10-031.

2000. Lower Skagit River Fecal Coliform Total Maximum Daily Load Submittal Report. Publication Number 00-10-010.

Wright, R.M., and A.J. McDonnell.

1979. In-stream Deoxygenation Rate Prediction. Journal Environmental Engineering Division, ASCE. 105(EE2). (Cited in EPA 1985 op.cit.)

APPENDIX A--PUBLIC INVOLVEMENT INFORMATION

The Department has tentatively determined to reissue a permit to the applicant listed on page one of this fact sheet. The permit contains conditions and effluent limitations which are described in the rest of this fact sheet.

Public Notice of Application (PNOA) was published on November 22, 1997, and November 29, 1997, in the *Skagit Valley Herald* to inform the public that an application had been submitted and to invite comment on the reissuance of this permit.

The Department will publish a Public Notice of Draft (PNOD) on date, in name of publication to inform the public that a draft permit and fact sheet are available for review. Interested persons are invited to submit written comments regarding the draft permit. The draft permit, fact sheet, and related documents are available for inspection and copying between the hours of 8:00 a.m. and 5:00 p.m. weekdays, by appointment, at the regional office listed below. Written comments should be mailed to:

Water Quality Permit Coordinator
Department of Ecology
Northwest Regional Office
3190 160th Avenue SE
Bellevue, WA 98008-5452

Any interested party may comment on the draft permit or request a public hearing on this draft permit within the thirty (30) day comment period to the address above. The request for a hearing shall indicate the interest of the party and the reasons why the hearing is warranted. The Department will hold a hearing if it determines there is a significant public interest in the draft permit (WAC 173-220-090). Public notice regarding any hearing will be circulated at least thirty (30) days in advance of the hearing. People expressing an interest in this permit will be mailed an individual notice of hearing (WAC 173-220-100).

The Department will consider all comments received within thirty (30) days from the date of public notice of draft indicated above, in formulating a final determination to issue, revise, or deny the permit. The Department's response to all significant comments is available upon request and will be mailed directly to people expressing an interest in this permit.

Further information may be obtained from the Department by telephone, 425-649-7215, or by writing to the address listed above.

This permit and fact sheet were written by Gerald Shervey, PE, Water Quality Engineer.

APPENDIX B--GLOSSARY

Acute Toxicity--The lethal effect of a pollutant on an organism that occurs within a short period of time, usually 48 to 96 hours.

AKART--An acronym for "all known, available, and reasonable methods of prevention, control, and treatment."

Ambient Water Quality--The existing environmental condition of the water in a receiving water body.

Ammonia--Ammonia is produced by the breakdown of nitrogenous materials in wastewater. Ammonia is toxic to aquatic organisms, exerts an oxygen demand, and contributes to eutrophication. It also increases the amount of chlorine needed to disinfect wastewater.

Average Monthly Discharge Limitation--The highest allowable average of daily discharges over a calendar month, calculated as the sum of all daily discharges measured during a calendar month divided by the number of daily discharges measured during that month (except in the case of fecal coliform). The daily discharge is calculated as the average measurement of the pollutant over the day.

Average Weekly Discharge Limitation--The highest allowable average of daily discharges over a calendar week, calculated as the sum of all daily discharges measured during a calendar week divided by the number of daily discharges measured during that week. The daily discharge is calculated as the average measurement of the pollutant over the day.

Best Management Practices (BMPs)--Schedules of activities, prohibitions of practices, maintenance procedures, and other physical, structural, and/or managerial practices to prevent or reduce the pollution of waters of the State. BMPs include treatment systems, operating procedures, and practices to control: plant site runoff, spillage or leaks, sludge or waste disposal, or drainage from raw material storage. BMPs may be further categorized as operational, source control, erosion and sediment control, and treatment BMPs.

BOD_5--Determining the Biochemical Oxygen Demand of an effluent is an indirect way of measuring the quantity of organic material present in an effluent that is utilized by bacteria. The BOD_5 is used in modeling to measure the reduction of dissolved oxygen in a receiving water after effluent is discharged. Stress caused by reduced dissolved oxygen levels makes organisms less competitive and less able to sustain their species in the aquatic environment. Although BOD is not a specific compound, it is defined as a conventional pollutant under the federal Clean Water Act.

Bypass--The intentional diversion of waste streams from any portion of a treatment facility.

Chlorine--Chlorine is used to disinfect wastewaters of pathogens harmful to human health. It is also extremely toxic to aquatic life.

Chronic Toxicity--The effect of a pollutant on an organism over a relatively long time, often 1/10 of an organism's life span or more. Chronic toxicity can measure survival, reproduction or growth rates, or other parameters to measure the toxic effects of a compound or combination of compounds.

Clean Water Act (CWA)--The Federal Water Pollution Control Act enacted by Public Law 92-500, as amended by Public Laws 95-217, 95-576, 96-483, 97-117; USC 1251 et seq.

Combined Sewer Overflow (CSO)--The event during which excess combined sewage flow caused by inflow is discharged from a combined sewer, rather than conveyed to the sewage treatment plant because either the capacity of the treatment plant or the combined sewer is exceeded.

Compliance Inspection - Without Sampling--A site visit for the purpose of determining the compliance of a facility with the terms and conditions of its permit or with applicable statutes and regulations.

Compliance Inspection - With Sampling--A site visit to accomplish the purpose of a Compliance Inspection - Without Sampling and as a minimum, sampling and analysis for all parameters with limits in the permit to ascertain compliance with those limits; and, for municipal facilities, sampling of influent to ascertain compliance with the percent removal requirement. Additional sampling may be conducted.

Composite Sample--A mixture of grab samples collected at the same sampling point at different times, formed either by continuous sampling or by mixing a minimum of four discrete samples. May be "time-composite" (collected at constant time intervals) or "flow-proportional" (collected either as a constant sample volume at time intervals proportional to stream flow, or collected by increasing the volume of each aliquot as the flow increased while maintaining a constant time interval between the aliquots).

Construction Activity--Clearing, grading, excavation, and any other activity which disturbs the surface of the land. Such activities may include road building, construction of residential houses, office buildings, or industrial buildings, and demolition activity.

Critical Condition--The time during which the combination of receiving water and waste discharge conditions have the highest potential for causing toxicity in the receiving water environment. This situation usually occurs when the flow within a water body is low, thus, its ability to dilute effluent is reduced.

Dilution Factor--A measure of the amount of mixing of effluent and receiving water that occurs at the boundary of the mixing zone. Expressed as the inverse of the effluent fraction, e.g., a dilution factor of 10 means the effluent comprises 10% by volume and the receiving water 90%.

Engineering Report--A document which thoroughly examines the engineering and administrative aspects of a particular domestic or industrial wastewater facility. The report shall contain the appropriate information required in WAC 173-240-060 or 173-240-130.

Fecal Coliform Bacteria--Fecal coliform bacteria are used as indicators of pathogenic bacteria in the effluent that are harmful to humans. Pathogenic bacteria in wastewater discharges are controlled by disinfecting the wastewater. The presence of high numbers of fecal coliform bacteria in a water body can indicate the recent release of untreated wastewater and/or the presence of animal feces.

Grab Sample--A single sample or measurement taken at a specific time or over as short period of time as is feasible.

Industrial User--A discharger of wastewater to the sanitary sewer which is not sanitary wastewater or is not equivalent to sanitary wastewater in character.

Industrial Wastewater--Water or liquid-carried waste from industrial or commercial processes, as distinct from domestic wastewater. These wastes may result from any process or activity of industry, manufacture, trade or business, from the development of any natural resource, or from animal operations such as feed lots, poultry houses, or dairies. The term includes contaminated storm water and, also, leachate from solid waste facilities.

Infiltration and Inflow (I/I)--"Infiltration" means the addition of ground water into a sewer through joints, the sewer pipe material, cracks, and other defects. "Inflow" means the addition of precipitation-caused drainage from roof drains, yard drains, basement drains, street catch basins, etc., into a sewer.

Interference--A discharge which, alone or in conjunction with a discharge or discharges from other sources, both:

Inhibits or disrupts the POTW, its treatment processes or operations, or its sludge processes, use or disposal; and

Therefore is a cause of a violation of any requirement of the POTW's NPDES permit (including an increase in the magnitude or duration of a violation) or of the prevention of sewage sludge use or disposal in compliance with the following statutory provisions and regulations or permits issued thereunder (or more stringent State or local regulations): Section 405 of the Clean Water Act, the Solid Waste Disposal Act (SWDA) [including title II, more commonly referred to as the Resource Conservation and Recovery Act (RCRA), and including State regulations contained in any State sludge management plan prepared pursuant to subtitle D of the SWDA], sludge regulations appearing in 40 CFR Part 507, the Clean Air Act, the Toxic Substances Control Act, and the Marine Protection, Research and Sanctuaries Act.

Major Facility--A facility discharging to surface water with an EPA rating score of >80 points based on such factors as flow volume, toxic pollutant potential, and public health impact.

Maximum Daily Discharge Limitation--The highest allowable daily discharge of a pollutant measured during a calendar day or any 24-hour period that reasonably represents the calendar day for purposes of sampling. The daily discharge is calculated as the average measurement of the pollutant over the day.

Method Detection Level (MDL)--The minimum concentration of a substance that can be measured and reported with 99% confidence that the analyte concentration is above zero and is determined from analysis of a sample in a given matrix containing the analyte.

Minor Facility--A facility discharging to surface water with an EPA rating score of <80 points based on such factors as flow volume, toxic pollutant potential, and public health impact.

Mixing Zone--A volume that surrounds an effluent discharge within which water quality criteria may be exceeded. The area of the authorized mixing zone is specified in a facility's permit and follows procedures outlined in State regulations (Chapter 173-201A WAC).

National Pollutant Discharge Elimination System (NPDES)--The NPDES (Section 402 of the Clean Water Act) is the Federal wastewater permitting system for discharges to navigable waters of the United States. Many states, including the State of Washington, have been delegated the authority to issue these permits. NPDES permits issued by Washington State permit writers are joint NPDES/State permits issued under both State and Federal laws.

Pass Through--A discharge which exits the POTW into waters of the State in quantities or concentrations which, alone or in conjunction with a discharge or discharges from other sources, is a cause of a violation of any requirement of the POTW's NPDES permit (including an increase in the magnitude or duration of a violation), or which is a cause of a violation of State water quality standards.

pH--The pH of a liquid measures its acidity or alkalinity. A pH of 7 is defined as neutral, and large variations above or below this value are considered harmful to most aquatic life.

Potential Significant Industrial User--A potential significant industrial user is defined as an Industrial User which does not meet the criteria for a Significant Industrial User, but which discharges wastewater meeting one or more of the following criteria:

a. Exceeds 0.5 % of treatment plant design capacity criteria and discharges <25,000 gallons per day; or

b. Is a member of a group of similar industrial users which, taken together, have the potential to cause pass through or interference at the POTW (e.g., facilities which develop photographic film or paper, and car washes).

The Department may determine that a discharger initially classified as a potential significant industrial user should be managed as a significant industrial user.

Quantitation Level (QL)--A calculated value five times the MDL (method detection level).

Significant Industrial User (SIU)--

1) All industrial users subject to Categorical Pretreatment Standards under 40 CFR 403.6 and 40 CFR Chapter I, Subchapter N; and

2) Any other industrial user that: discharges an average of 25,000 gallons per day or more of process wastewater to the POTW (excluding sanitary, non-contact cooling, and boiler blow-down wastewater); contributes a process wastestream that makes up 5 percent or more of the average dry weather hydraulic or organic capacity of the POTW treatment plant; or is designated as such by the Control Authority* on the basis that the industrial user has a reasonable potential for adversely affecting the POTW's operation or for violating any pretreatment standard or requirement [in accordance with 40 CFR 403.8(f)(6)].

Upon finding that the industrial user meeting the criteria in paragraph 2, above, has no reasonable potential for adversely affecting the POTW's operation or for violating any pretreatment standard or requirement, the Control Authority* may at any time, on its own initiative or in response to a petition received from an industrial user or POTW, and in accordance with 40 CFR 403.8(f)(6), determine that such industrial user is not a significant industrial user.

*The term "Control Authority" refers to the Washington State Department of Ecology in the case of non-delegated POTWs or to the POTW in the case of delegated POTWs.

State Waters--Lakes, rivers, ponds, streams, inland waters, underground waters, salt waters, wetlands, and all other surface waters and watercourses within the jurisdiction of the state of Washington.

Stormwater--That portion of precipitation that does not naturally percolate into the ground or evaporate, but flows via overland flow, inter-flow, pipes, and other features of a storm water drainage system into a defined surface water body, or a constructed infiltration facility.

Technology-based Effluent Limit--A permit limit that is based on the ability of a treatment method to reduce the pollutant.

Total Suspended Solids (TSS)--Total suspended solids are the particulate materials in an effluent. Large quantities of TSS discharged to a receiving water may result in solids accumulation. Apart from any toxic effects attributable to substances leached out by water, suspended solids may kill fish, shellfish, and other aquatic organisms by causing abrasive injuries and by clogging the gills and respiratory passages of various aquatic fauna. Indirectly, suspended solids can screen out light and can promote and maintain the development of noxious conditions through oxygen depletion.

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

Water Quality-based Effluent Limit--A limit on the concentration or mass of an effluent parameter that is intended to prevent the concentration of that parameter from exceeding its water quality criterion after it is discharged into a receiving water.

APPENDIX C--TECHNICAL CALCULATIONS

Several of the Excel® spreadsheet tools used to evaluate a discharger's ability to meet Washington State water quality standards can be found on the Department's homepage at http.www:wa.gov.ecology.

Tables 11 and 12 show the estimate of dilution provided during critical conditions near the outlet of the WWTP outfall in the Skagit River. The table reflects the calculations for the spread of a plume from a point source in a river with boundary effects from the shoreline based on the method of Fischer et al. (1979) with correction for the effective origin of effluent. The calculations are based on the procedures and formulas cited in Mixing in Inland and Coastal Waters by Fischer, et al., 1979. New York: Academic Press. This procedure is also cited by EPA in the Technical Support Document for Water Quality-based Toxics Control, U.S. EPA, March 1991 (EPA/505/2-90-001).

Table 13: Chronic Dilution Estimation from Fischer.

	calibrate	1999 flows	2015 flow
INPUT			
1. Effluent Discharge Rate (MGD):	3.2	3.5	8.1
1. Effluent Discharge Rate (cfs):	4.93	5.41	12.52
2. Receiving Water Characteristics Downstream From Waste Input			
Stream Depth (ft):	8.00	6.75	6.75
Stream Velocity (fps):	1.40	1.10	1.10
Channel Width (ft):	400.00	400	400
Stream Slope (ft/ft) or Manning roughness "n":	0.029	0.029	0.029
0 if slope or 1 if Manning "n" in previous cell:	1	1	1
3. Discharge Distance From Nearest Shoreline (ft):	14	12	12
4. Location of Point of Interest to Estimate Dilution			
Distance Downstream to Point of Interest (ft):	200	306	306
Distance From Nearest Shoreline (ft):	14	5	5
5. Transverse Mixing Coefficient Constant (usually 0.6):	0.6	0.6	0.6
6. Original Fischer Method (enter 0) or *Effective Origin* Modification (enter 1)	0	0	0
OUTPUT			
1. Source Conservative Mass Input Rate			
Concentration of Conservative Substance (%):	100.00	100.00	100.00
Source Conservative Mass Input Rate (cfs*%):	493.00	541.00	1,252.00
2. Shear Velocity			
Shear Velocity based on slope (ft/sec):	#N/A	#N/A	#N/A
Shear Velocity based on Manning "n":			
using Prasuhn equations 8-26 and 8-54 assuming			
hydraulic radius equals depth for wide channel			
Darcy-Weisbach friction factor "f":	0.049	0.052	0.052
Shear Velocity from Darcy-Weisbach "f" (ft/sec):	0.109	0.088	0.088
Selected Shear Velocity for next step (ft/sec):	0.109	0.088	0.088
3. Transverse Mixing Coefficient (ft2/sec):	0.525	0.358	0.358
4. Plume Characteristics Accounting for Shoreline Effect (Fischer *et al.*, 1979)			
Co	1.10E-01	1.82E-01	4.22E-01
x'	4.69E-04	6.22E-04	6.22E-04
y'o	3.50E-02	3.00E-02	3.00E-02
y' at point of interest	3.50E-02	1.25E-02	1.25E-02
Solution using superposition equation (Fischer eqn 5.9)			
Term for n= -2	0.00E+00	0.00E+00	0.00E+00
Term for n= -1	0.00E+00	0.00E+00	0.00E+00
Term for n= 0	1.07E+00	1.37E+00	1.37E+00
Term for n= 1	0.00E+00	0.00E+00	0.00E+00
Term for n= 2	0.00E+00	0.00E+00	0.00E+00
Upstream Distance from Outfall to *Effective Origin* of Effluent Source (ft)	#N/A	#N/A	#N/A
Effective Distance Downstream from Effluent to Point of Interest (ft)	200.00	306.00	306.00
x' Adjusted for *Effective Origin*	4.69E-04	6.22E-04	6.22E-04
C/Co (dimensionless)	1.40E+01	1.55E+01	1.55E+01
Concentration at Point of Interest (Fischer Eqn 5.9)	1.54E+00	2.82E+00	6.52E+00
Unbounded Plume Width at Point of Interest (ft)	48.980	56.445	56.445
Unbounded Plume half-width (ft)	24.490	28.222	28.222
Distance from near shore to discharge point (ft)	14.00	12.00	12.00
Distance from far shore to discharge point (ft)	386.00	388.00	388.00
Plume width bounded by shoreline (ft)	38.49	40.22	40.22
Approximate Downstream Distance to Complete Mix (ft):	158,990	185,074	185,074
Theoretical Dilution Factor at Complete Mix:	908.722	548.983	237.220
Calculated Flux-Average Dilution Factor Across Entire Plume Width:	87.442	55.204	23.854
Calculated Dilution Factor at Point of Interest:	65	35	15
Field measured dilution	65		

Table 14: Acute Dilution Estimation from Fischer.

[redacted]

	calibrate	1999 flow	2015 flow
1. Effluent Discharge Rate (MGD):	3.2	6.4	
1. Effluent Discharge Rate (cfs):	4.93	9.90	19.80
2. Receiving Water Characteristics Downstream From Waste Input			
Stream Depth (ft):	8.00	6.75	6.75
Stream Velocity (fps):	1.30	1.20	1.20
Channel Width (ft):	400.00	400.00	400.00
Stream Slope (ft/ft) or Manning roughness "n":	0.021	0.021	0.021
0 if slope or 1 if Manning "n" in previous cell:	1	1	1
3. Discharge Distance From Nearest Shoreline (ft):	14	14	14
4. Location of Point of Interest to Estimate Dilution			
Distance Downstream to Point of Interest (ft):	31	31	31
Distance From Nearest Shoreline (ft):	14	14	14
5. Transverse Mixing Coefficient Constant (usually 0.6):	0.23	0.23	0.23
6. Original Fischer Method (enter 0) or *Effective Origin* Modification (enter 1)	0	0	0

[redacted]

1. Source Conservative Mass Input Rate			
Concentration of Conservative Substance (%):	100.00	100.00	100.00
Source Conservative Mass Input Rate (cfs*%):	493.00	990.00	1,980.00
2. Shear Velocity			
Shear Velocity based on slope (ft/sec):	#N/A	#N/A	#N/A
Shear Velocity based on Manning "n":			
using Prasuhn equations 8-26 and 8-54 assuming			
hydraulic radius equals depth for wide channel			
Darcy-Weisbach friction factor "f":	0.026	0.027	0.027
Shear Velocity from Darcy-Weisbach "f" (ft/sec):	0.074	0.070	0.070
Selected Shear Velocity for next step (ft/sec):	0.074	0.070	0.070
3. Transverse Mixing Coefficient (ft2/sec):	0.135	0.108	0.108
4. Plume Characteristics Accounting for Shoreline Effect (Fischer *et al.*, 1979)			
Co	1.19E-01	3.06E-01	6.11E-01
x'	2.02E-05	1.75E-05	1.75E-05
y'o	3.50E-02	3.50E-02	3.50E-02
y' at point of interest	3.50E-02	3.50E-02	3.50E-02
Solution using superposition equation (Fischer eqn 5.9)			
Term for n= -2	0.00E+00	0.00E+00	0.00E+00
Term for n= -1	0.00E+00	0.00E+00	0.00E+00
Term for n= 0	1.00E+00	1.00E+00	1.00E+00
Term for n= 1	0.00E+00	0.00E+00	0.00E+00
Term for n= 2	0.00E+00	0.00E+00	0.00E+00
Upstream Distance from Outfall to *Effective Origin* of Effluent Source (ft)	#N/A	#N/A	#N/A
Effective Distance Downstream from Effluent to Point of Interest (ft)	31.00	31.00	31.00
x' Adjusted for *Effective Origin*	2.02E-05	1.75E-05	1.75E-05
C/Co (dimensionless)	6.28E+01	6.74E+01	6.74E+01
Concentration at Point of Interest (Fischer Eqn 5.9)	7.45E+00	2.06E+01	4.12E+01
Unbounded Plume Width at Point of Interest (ft)	10.160	9.465	9.465
Unbounded Plume half-width (ft)	5.080	4.733	4.733
Distance from near shore to discharge point (ft)	14.00	14.00	14.00
Distance from far shore to discharge point (ft)	386.00	386.00	386.00
Plume width bounded by shoreline (ft)	10.16	9.47	9.47
Approximate Downstream Distance to Complete Mix (ft):	572,758	659,871	659,871
Theoretical Dilution Factor at Complete Mix:	843.813	327.273	163.636
Calculated Flux-Average Dilution Factor Across Entire Plume Width:	21.433	7.744	3.872
Calculated Dilution Factor at Point of Interest:	13	5	2
Field measured dilution	13		

Statistical analysis of ammonia concentrations measured at the WWTP for the last 5 years.

Ammonia data, statistical evaluation, and limit calculations are shown below. Ammonia limitations were derived using normal statistics (normal shaped bell curve). The histogram shows the tendency of ammonia concentration sample results to be normally distributed – the data peak is in the center of the distribution and 'tails' off on either end of the distribution. Data submitted by the Permittee is listed in Table 13.

Statistics for normal distribution of ammonia concentrations	
Mean	22.77
Standard Error	0.65
Median	23
Mode	24
Standard Deviation (SD)	**6.0**
Sample Variance (V)	**36.3**
Kurtosis	-0.48
Skewness	0.15
Range	29
Minimum	9
Maximum	38
Z_{99}	2.326
Z_{95}	1.645

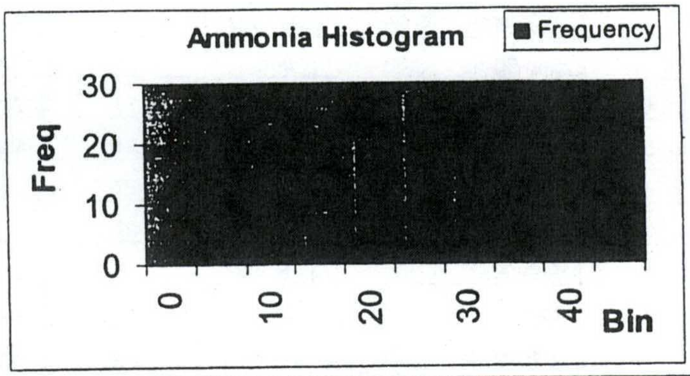

Figure 5: Frequency histogram for ammonia exhibits a bell shaped curve shape, distribution of ammonia data follows the normal distribution, not the lognormal. The data is shown at the end of this section.

Effluent ammonia concentrations follow a normal distribution; Figure 4 shows that the data are symmetrically distributed around the mean. Permit limit calculations using a lognormal distribution do not model the effluent at well as one using a normal distribution. Based on the procedure by EPA, 1991, in Appendix E-5, the derivation of permit limitations for data that is normally distributed is as follows.

From the Water Quality-Based Permit Limitations Spreadsheet (Table 8), the waste load allocations for ammonia are WLA_a=41, WLA_c=65. The long term averages (LTA) needed to meet the WLAs are: $LTA_a = WLA_a - (Z_{99} \times SD) = 41 - (2.326 \times 6) = 27$; for $LTA_c = WLA_c - (Z_{99} \times SD) = 65 - (2.326 \times 6) = 51$. The acute LTA is more limiting. The maximum daily limit is equal to the acute WLA = **MDL = 41 mg/L**. The average monthly limit, the average of all individual values measured for the month that should not be exceeded to assure that the WLA is not exceeded is **AML** = LTA_a + (Variance/# samples per month)$^{1/2} \times Z_{95} = 27 + (36.3/9)^{1/2} \times 1.645 =$ **30 mg/L.**

Table 15: Ammonia Data from the WWTP.

Month	Concentration (mg/L)	Month	Concentration (mg/L)	Month	Concentration (mg/L)	Month	Concentration (mg/L)
Mar-93	16.4	Feb-95	20	Nov-96	21	Aug-98	24
Apr-93	16.6	Mar-95	29	Dec-96	13	Sep-98	31
May-93	17.3	Apr-95	25	Jan-97	9	Oct-98	23
Jun-93	22.9	May-95	26	Feb-97	19	Nov-98	18
Jul-93	21.7	Jun-95	33	Mar-97	17	Dec-98	18
Aug-93	24.9	Jul-95	34	Apr-97	26	Jan-99	16
Sep-93	28.1	Aug-95	35	May-97	27	Feb-99	16
Oct-93	25.5	Sep-95	38	Jun-97	24	Mar-99	21
Nov-93	28.1	Oct-95	16	Jul-97	16	Apr-99	24
Dec-93	24.5	Nov-95	19	Aug-97	24	May-99	17
Jan-94	21.4	Dec-95	21	Sep-97	27	Jun-99	23
Mar-94	22.3	Jan-96	13	Oct-97	27	Jul-99	15
May-94	31.8	Feb-96	17	Nov-97	18	Aug-99	23
Jun-94	29.4	Mar-96	26	Dec-97	13	Sep-99	30
Jul-94	22	Apr-96	21	Jan-98	19	Oct-99	30
Aug-94	33.3	May-96	24	Feb-98	27	Nov-99	18
Sep-94	17.6	Jun-96	31	Mar-98	13	Dec-99	14
Oct-94	24.5	Jul-96	31	Apr-98	15	Jan-00	22
Nov-94	21	Aug-96	31	May-98	14	Feb-00	26
Dec-94	16	Sep-96	28	Jun-98	23	Mar-00	23
Jan-95	24	Oct-96	25	Jul-98	22	Apr-00	26
						May-00	31

Statistical analysis of copper concentrations

date	tot. rec. copper (ug/L)	Ln
7/14/1993	11	2.40
7/27/1993	19	2.94
8/9/1993	11	2.40
8/22/1993	10	2.30
9/4/1993	17	2.83
9/17/1993	13	2.56
10/13/1993	23	3.14
10/26/1993	27	3.30
11/8/1993	27	3.30
11/21/1993	25	3.22
12/4/1993	15	2.71
12/17/1993	27	3.30
1/12/1994	20	3.00
1/25/1994	14	2.64
2/7/1994	23	3.14
2/20/1994	13	2.56
3/5/1994	13	2.56
3/18/1994	16	2.77
3/31/1994	16.9	2.83
4/13/1994	14	2.64
4/26/1994	20	3.00
5/9/1994	28	3.33
5/22/1994	20	3.00
6/17/1994	28	3.33
6/30/1994	34	3.53
7/5/1995	21	3.04
7/10/1995	20	3.00
7/18/1995	10.6	2.36
7/25/1995	18.9	2.94
8/5/1994	10.6	2.36
8/15/1995	10.9	2.39
9/16/1994	27.1	3.30
9/29/1994	13	2.56
10/12/1994	10	2.30
10/25/1994	17.6	2.87
11/7/1994	22.3	3.10
11/20/1994	10	2.30
12/16/1994	10	2.30
12/29/1994	10	2.30
1/11/1995	10.6	2.36
1/24/1995	13.7	2.62

Mt Vernon Effluent Copper data

Statistical output for logarithmic distribution

	ln	antilog
Mean	2.80	17.62
Standard Error	0.06	
coeff variation	0.13	**0.38**
Median	2.83	
Mode	2.30	
Standard Deviation	0.37	
Sample Variance	0.14	42.38
Kurtosis	-1.26	
Skewness	0.13	
Range	1.22	
Minimum	2.30	
Maximum	3.53	
Sum	114.83	
Count	41.00	
Confidence Level(95.000%)	0.11	

Raw Data Summary	
min	10
max	34
Count	41

Bin	Frequenc
10 to	5
14 to	13
18 to	5
22 to	7
26 to	4
30 to	6
34 to	1
38	0

Expected max concentrations based on lognormal distribution

Confidence interval	Total recov. max conc.	dissolved max conc.
95% CI	30.2	26.0
96% CI	31.4	27.1
97% CI	33.0	28.4
98% CI	**35.1**	30.3
99% CI	38.9	33.5

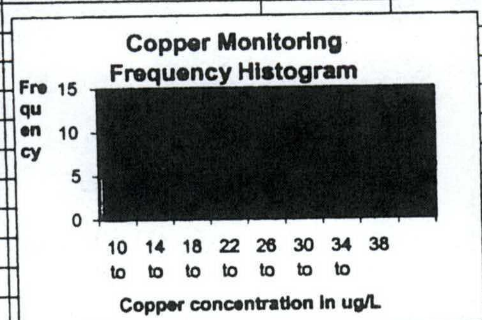

Table 16: Results of copper testing and statistical analysis. The 98% maximum of 35 mg/L and CV of 0.38 were used in Table 15 to calculate interim limits. The frequency histogram shows the asymmetrical shape of a lognormal distribution. The maximum value for permit limitations selected was 35 ug/L and was based on BPJ of the author. It is slightly higher than the maximum measured, but is the maximum value predicted for the data at 98% confidence.

Table 17: This spreadsheet calculates water quality-based permit limits based on the two value steady state model using the State Water Quality Standards contained in WAC 173-201A. The procedure and calculations are done per the procedure in Technical Support Document for Water Quality-based Toxics Control, U.S. EPA, March 1991 (EPA/505/2-90-001) on page 99.

	Acute Dil'n Factor	Chronic Dil'n Factor	Metal Criteria Translator	Metal Criteria Translator	Ambient Concentration	Water Quality Standard Acute	Water Quality Standard Chronic	Average Monthly Limit (AML)	Maximum Daily Limit (MDL)	Comments	WLA Acute	WLA Chronic	LTA Acute	LTA Chronic	LTA Coeff. Var. (CV)	Limiting LTA	Coeff. Var. (CV)	# of Samples per Month
Parameter			Acute	Chronic	ug/L	ug/L	ug/L	ug/L	ug/L		ug/L	ug/L	ug/L	ug/L	decimal	ug/L	decimal	n
NH3-N (in mg/L)	5	35			0.023	8.31	1.88	18.4	41.5	not used	41	65.02	13.3	34.3	0.60	13.3	0.60	8.00
copper	5	35	1.00	1.00	0.5500	4.61	3.47	12.8	20.9		21	102.82	9.5	67.5	0.38	9.5	0.38	4.00
zinc	5	35	1.00	1.00	0.0200	35.36	32.29	88.4	177.4		177	1129.35	56.7	595.7	0.60	56.7	0.60	4.00
chlorine	5	35				19.00	11.00	47.4	95.0		95	385.00	30.5	203.1	0.60	30.5	0.60	4.00
NH3-N (in mg/L)	10	35			0.023	8.31	1.88	41.3	82.9	informational	83	65.02	26.6	34.3	0.60	26.6	0.60	4.00
copper	10	35	1.00	1.00	0.5500	4.61	3.47	18.6	32.8	informational	41	102.82	13.2	54.2	0.60	13.2	0.45	4.00
mercury	10	35	0.85	0.85		2.10	0.012	0.4	0.8	informational	21	0.42	6.7	0.2	0.60	0.2	0.60	4.00
lead	10	35	0.47	0.47		13.88	0.54	33.3	66.7	informational	139	18.93	44.6	10.0	0.60	10.0	0.60	4.00
silver	10	35	0.85			0.32	1000.00	1.9	3.8	informational	3	35000.00	1.0	18460.2	0.60	1.0	0.60	4.00
zinc	10	35	1.00	1.00	0.0200	35.36	32.29	176.9	354.8	informational	353	1129.35	113.5	595.7	0.60	113.5	0.60	4.00
Copper (interim)								**21.3**	**35.0**		35	1000	15.9	656.9	0.38	15.9	0.38	

Table 18: Calculation of pH of a mixture of two flows. Ecology model PH-MIX.WK1. Based on the procedure in EPA's DESCON program (EPA, 1988. Technical Guidance on Supplementary Stream Design Conditions for Steady State Modeling. USEPA Office of Water, Washington, D.C.)

Input data is taken from the Lower Skagit TMDL Data Summary (Ecology Report #96-345), Mount Vernon WWTP DMRs, the Mount Vernon WWTP Mixing Zone Study (Cosmospolitan Engineering Group, 2000), and the Department's ambient monitoring records from 1994 to present for the Skagit River.					try 100:1 dilution	comments
INPUT *********	range	**********	**********	***********		
1. UPSTREAM CHARACTERISTICS						
Upstream Discharge (cfs) [at 35:1 dil, effluent Q * 35]		122.5	122.5	122.5	350.0	
Upstream Temperature (deg C)	1.4 to 15.4	15.2	15.2	15.2	15.2	90%ile from Cosmopolitan, 2000
Upstream pH	8.1 to 6.7	7.7	7.7	6.7	7.7	90%ile from Cosmopolitan, 2000
Upstream Alkalinity (mg CaCO3/L)	20 to 25	20.0	20.0	20.0	20.0	
2. EFFLUENT CHARACTERISTICS						
Effluent Discharge (cfs)		3.5	3.5	3.5	3.5	
Effluent Temperature (deg C)		20.0	20.0	20.0	20.0	lower skagit TMDL dry season
Effluent pH	6.1 to 7.8	**6.6**	**6.5**	**9.0**	**6.2**	DMRs
Effluent Alkalinity (mg CaCO3/L)		195.0	195.0	195.0	195.0	1 sample, lower skagit TMDL
OUTPUT *********						
1. IONIZATION CONSTANTS						
Upstream pKa		6.4	6.4	6.4	6.4	
Effluent pKa		6.4	6.4	6.4	6.4	
2. IONIZATION FRACTIONS						
Upstream Ionization Fraction		1.0	1.0	0.7	1.0	
Effluent Ionization Fraction		0.6	0.6	1.0	0.4	
3. TOTAL INORGANIC CARBON						
Upstream Total Inorganic Carbon (mg CaCO3/L)		21.0	21.0	30.5	21.0	
Effluent Total Inorganic Carbon (mg CaCO3/L)		313.0	343.6	195.5	491.5	
4. DOWNSTREAM MIXED FLOW CONDITIONS						
Mixture Temperature (deg C)		15.3	15.3	15.3	15.2	
Mixture Alkalinity (mg CaCO3/L)		24.9	24.9	24.9	21.7	
Mixture Total Inorganic Carbon (mg CaCO3/L)		29.2	30.0	35.0	25.7	
Mixture pKa		6.4	6.4	6.4	6.4	
pH of Mixture		7.2	7.1	6.8	7.2	
change at edge of mixing zone limited to 0.5		**0.5**	**0.6**	**-0.1**	**0.5**	
Calculations predict that pH lower than 6.6 violates WQ standard by inducing change greater than 0.5						

Table 19: This spreadsheet calculates the reasonable potential to exceed State Water Quality Standards for a small number of samples. The procedure and calculations are done per the procedure in Technical Support Document for Water Quality-based Toxics Control, U.S. EPA, March 1991 (EPA/505/2-90-001) on page 56.

This spreadsheet calculates the reasonable potential to exceed state water quality standards for a small number of samples. The procedure and calculations are done per the procedure Technical Support Document for Water Quality-based Toxics Contr U.S. EPA, March, 1991 (EPA/505/2-90-001) on page 56. User input columns are shown with red headings. Corrected formulas in col G and H on 5/98 (GB)

				State Water Quality Standard		Max concentration at edge of...			CALCULATIONS								
	Metal Criteria Translator as decimal	Metal Criteria Translator as decimal	Ambient Concentration (metals as dissolved)	Acute	Chronic	Acute Mixing Zone	Chronic Mixing Zone	LIMIT REQ'D?	Effluent percentile value		Max effluent conc. measured (metals as total recoverable)	Coeff Variation		# of samples	Multiplier	Acute Dil'n Factor
Parameter	Acute	Chronic	ug/L	ug/L	ug/L	ug/L	ug/L			Pn	ug/L	CV	s	n		
NH3-N (in mg/L)			0.0150	8.31	1.8800	8.81	1.27	YES	0.95	0.972	44.00	0.60	0.55	104	1.00	5
copper	1.00	1.00	0.5500	4.61	3.47	7.81	1.59	YES	0.95	0.957	37.00	0.45	0.43	68	1.00	5
mercury	0.85	0.85		2.10	0.012	0.31	0.04	YES	0.95	0.717	1.00	0.60	0.55	9	1.81	5
lead	0.47	0.47	0.0200	13.88	0.54	0.35	0.07	NO	0.95	0.717	2.00	0.60	0.55	9	1.81	5
silver	0.85			0.32	1000.00	6.16	1.04	YES	0.95	0.717	20.00	0.60	0.55	9	1.81	5
zinc	1.00	1.00	9.3000	35.36	32.29	46.41	14.60	YES	0.95	0.717	108.00	0.60	0.55	9	1.81	5
BHC - GAMMA 58899 4P (Lindane)				2.0	0.08	0.02	0.00	NO	0.95	0.717	0.06	0.60	0.55	9	1.81	5
BIS(2-ETHYLHEXYL) PHTHALATE 117817 13B				940.0	3.0	15.22	2.17	NO	0.95	0.717	42.00	0.60	0.55	9	1.81	5
CHLOROFORM 67663 11V				28900.0	1240.0	1.12	0.16	NO	0.95	0.717	3.10	0.60	0.55	9	1.81	5
CYANIDE 57125 14M				22.0	5.2	9.42	1.35	NO	0.95	0.717	26.00	0.60	0.55	9	1.81	5

Values entered for mercury, lead, and silver are the detection limit at the laboratory performing the analysis. These analytes were not detected in the effluent. If present, they are at levels below these values. No permit limit is derived for these analytes.

APPENDIX D--ORDER ON CONSENT NO. DE 96WQ-N105

The text of the Order on Consent No. DE 96WQ-N105 follows. The order was effective on April 11, 1996. It was signed by the Honorable Skye Richendrfer, Mayor, City of Mount Vernon and John H Glynn, Supervisor, Water Quality, Washington Department of Ecology Northwest Regional Office.

STATE OF WASHINGTON

DEPARTMENT OF ECOLOGY

THE MATTER OF THE COMPLIANCE BY	)	
THE CITY OF MOUNT VERNON	)	
with Chapter 90.48 RCW and the Rules and	)	ORDER ON CONSENT
Regulations of the Department of Ecology	)	No. DE 96WQ-N105

I. INTRODUCTION

This order is issued to the City of Mount Vernon (the City) by the State of Washington Department of Ecology (the Department), pursuant to Chapter 90.48 of the Revised Code of Washington (RCW), otherwise known as the Water Pollution Control Act. The RCW 90.48.260 designates the Department as the state water pollution control agency for all purposes of the Federal Clean Water Act (CWA) and grants complete authority to administer a National Pollutant Discharge Elimination System (NPDES) permit program. The Department is authorized to issue permits, which include effluent treatment and limitation requirements, as well as inspection, monitoring, and reporting requirements. The Department may terminate or modify permits as well as perform enforcement.

This order is being issued with the cooperation of the City to fulfill the legal requirements of Chapter 173-245 of the Washington Administrative Code (WAC). The purpose of those requirements is to reduce the environmental impact of combined stormwater and raw sewage discharges on the waters of the State of Washington.

II. FACTS REGARDING THIS CASE

A. On June 16, 1993, the Department of Ecology issued NPDES Permit No. WA-022407-4 to the City of Mount Vernon.

B. Chapter 90.48.480 of the RCW requires the submittal of a plan and schedule to achieve the greatest reasonable reduction of combined sewer overflows (CSOs) at the earliest possible date. *The City of Mount Vernon Comprehensive Sewer and Combined Sewer Overflow Reduction Plan (The Plan)* was submitted in April of 1991. *The Plan* met this requirement and was approved by the Department on June 20 of 1995. *The Plan* was adopted by the City Council in December of 1995.

C. Chapter 173-245 of Washington Administrative Code (WAC) defines the "greatest reasonable reduction of combined sewer overflows" as limiting the frequency of untreated CSOs to an average of no more than one per year. In approving *The Plan*, the Department has determined that the City will be making a good faith effort to achieve this goal. If the current standards are revised, this Consent Order may be renegotiated if requested by either party.

D. Special Condition S11. of NPDES Permit No. WA-022407-4 requires the submittal of annual CSO Reports to the Department, in accordance with WAC 173-245-090.

E. *The Plan* provides for both in-line and reservoir storage of CSO flows prior to treatment at the Mount Vernon Wastewater Treatment Plant. The plan requires construction of an oversized sewer interceptor line to transport and store the majority of CSO flows within the next four years and construction of additional treatment capacity at the treatment plant by the year 2015. Final design of the treatment facilities is dependent on installation of monitoring equipment in the sewer interceptor line.

F. The transfer of CSO flows to the Mount Vernon Wastewater Treatment Plant (WWTP) will dilute the biochemical oxygen demand (BOD) and total suspended solids (TSS) concentration of the influent. This may prevent the WWTP from meeting the current NPDES requirement to remove 85% of these pollutants from the incoming wastestream.

G. The transfer of CSO flows to the Mount Vernon WWTP will result in a substantial increase in grit entering the facility. The increase in grit may result in a reduction in the calculated percentage reduction in volatile solids in digested sludge resulting in failure to meet vector attraction reduction requirements.

H. The increase in flow through the plant will result in decreased chlorination contact time in the WWTP.

I. The Department may relax the NPDES permit limits for percent removal requirements for conventional pollutants based on engineering submittals and in accordance with State and Federal regulations related to dilute influents and standards for CSO reduction and treatment.

J. The increase in flow through the plant is expected to exceed the 85%-of-design, wet weather design flow during wet weather months.

K. The City is proceeding to comply with permit condition S4.B, Plans for Maintaining Adequate Capacity, by submitting an engineering report that assesses the existing capacity of the Mount Vernon WWTP and identifies improvements to increase the capacity sufficiently to treat increased CSO flows to the plant. The draft report entitled *Mount Vernon Wastewater Treatment Plant Evaluation* dated July 1995, was submitted for comment on October 26, 1995. The City will have satisfied permit condition S4.B for the additional flows from CSO transport as they are currently estimated and condition S4.D, Capacity Assessment, when the Department approves the final report.

III. COMPLETION SCHEDULE FOR CSO REDUCTION

The City of Mount Vernon shall reduce the combined sewer overflows in its wastewater collection system as outlined in Table I-1 of *the Plan* and described as "Related to CSO Reduction." Work shall be completed as follows:

A. Complete construction of the central interceptor (Improvement numbers CA-1b, CA-1-c, CA-1-d, October 1994 revision) no later than December 31, 2000.

B. Provide modulating controller for the motor operated sluice gate at the manhole upstream from the wastewater treatment plant influent pump station (Improvement number T-2) no later than December 31, 2000.

C. Upgrade the Park Street Pump Station with new pumps (Improvement number CA-1a) no later than January 1, 2015.

D. Upgrade wastewater treatment plant to provide treatment/storage of CSO flows to reduce overflow events to an average of one per year no later than January 1, 2015.

IV. MONITORING AND REPORTING

A. Estimate the frequency and volume of CSO discharges from the Park Street and Division Street Pump Stations until new monitoring devices are installed during construction of the central interceptor projects.

B. Record frequency and volume of CSO discharges as soon as construction of portions of the Central interceptor allow. Direct recording of frequency and volume of CSO discharges shall commence no later than December 31, 2000.

C. Provide annual CSO reports for the previous calendar year to the Department of Ecology no later than March 31 each year. The reports shall comply with the requirements of WAC 173-245-090 (1):

   1. Detail the past year's frequency and volume of combined sewage discharged from each CSO discharge site.

   2. Explain the CSO reduction accomplishments of the previous year.

   3. List the projects associated with CSO reduction planned for the next year.

   4. If there is an increase in the annual baseline volume or frequency of CSO discharges described in *the Plan* (Figure 5-16), then the City shall propose a project and schedule to reduce that CSO site or group of sites to or below the baseline condition.

D. In conjunction with the City's application for renewal of its NPDES permit, provide an amendment to its CSO Reduction Plan addressing the elements of WAC 173-245-090(2).

V. FACILITY OPERATION

A. The wastewater treatment plant, sewage collection system, pump stations, and all other facilities associated with wastewater collection, treatment, and discharge shall be operated so as to minimize the discharge of untreated combined sewage to waters of the state while meeting the conditions of NPDES permit number WA-002407-4.

B. No discharge of sanitary sewage as defined WAC 173-245 shall occur from CSO outfalls during dry weather.

C. Discharges from the CSO outfalls shall not exceed the baseline values described in *the Plan* (Figure 5-16).

VI. NOTIFICATION OF NONCOMPLIANCE

The City shall promptly notify Ecology of any occurrence which may result in noncompliance with the requirements of the Consent Order which is caused by circumstances beyond the City's control which could not be overcome by due diligence. Such notification shall state the nature of the anticipated noncompliance, the reasons therefore, the expected duration of the noncompliance and any mitigating actions taken.

VII. JUDICIAL REVIEW

In the event that the City fails, without sufficient cause, to comply with any of the terms of this Consent Order, the Order may be enforced pursuant to the powers vested in Ecology by law. No party, other than Ecology, will bring any legal action except as allowed herein to appeal, challenge, or construe any portion of this Consent Order.

VIII. STIPULATIONS

By the signatures appearing below, the City of Mount Vernon hereby consents and agrees to:

A. The issuance of the Order;

B. Perform and comply with the City's obligations as specified in the Order; and

Not appeal, contest, or legally challenge the issuance of the Consent Order or Ecology's jurisdiction to enforce this Consent Order.

APPENDIX E--RESPONSE TO COMMENTS

DISCHARGE MONITORING REPORT (DMR) INSTRUCTIONS

To avoid processing delays and the need to resubmit your DMR's, please comply with the following requirements:

- Enter the monitoring period at the top of the form. Monitoring periods consist of a calendar month or months (quarterly reporting). (For example, July 1-July 31, not June 27-July 27)

- The forms must be received at the Department of Ecology Northwest Regional Office by the date specified in your permit. Address the envelope to the attention of Chris Smith, WPLCS Coordinator, 3190 160th Avenue SE, Bellevue, WA 98008-5452.

- All entries on the forms must be in ink or typewritten. The forms must be signed in ink by the responsible official for the facility or by a person who has been designated authority to do so in writing by the responsible official. The Department must have a record of the designation letter on file to accept signatures by persons other than the responsible official.

- Circle permit violations and provide a written explanation of the cause of the violation and remedies used to correct the problem. The number of violations must be entered on the DMR form under the "No. Ex" column on the right side of the DMR form. See the instructions on the back of the DMR form for details on how to fill in that column.

- Failure to report the results of tests required by your permit is a permit violation. If your facility did not discharge during the monitoring period, indicate by checking the box in the upper right hand corner for no discharge. Items that are not required for the monitoring period (such as tests done once per quarter) should be labeled "NA" for not applicable.

If you encounter difficulty using the enclosed form, contact your facility manager. Enclosed are double sided forms. Keep at least one blank form to photocopy. You are responsible for keeping forms on hand for use at your facility.

Questions; contact Chris Smith, WPLCS Coordinator, (425) 649-7214.

Permit[illegible]ne/Address
Include Name/Location (if different)

NAME CITY OF MOUNT VERNON #665
ADDRESS POB 809
MOUNT VERNON WA 98273
FACILITY WASTEWATER TREATMENT PLANT
LOCATION 1401 BRITT ROAD

NATIONAL P[illegible]NT DISCHARGE ELIMINATION SYSTEM
DISCHARGE MONITORING REPORT(*DMR*)

WA-002407-4	001
PERMIT NUMBER	DISCHARGE NUMBER

	MONITORING PERIOD						
	YEAR	MO	DAY		YEAR	MO	DAY
FROM				TO			

NOTE: Read in[illegible]efore completing this fo[illegible]...

Discharge Location	
Lat	48° 24' 48" N
Long	122° 20' 6" W
NO DISCHARGE	

Parameter		QUANTITY OR LOADING Average	Maximum	Units	QUALITY OR CONCENTRATION Minimum	Average	Maximum	Units	No. of Exceed-ances	Frequency of Analysis	Sample Type
FLOW	Sample Measurement			MGD	*******	*******	*******	***		07/07	CONT.
	Permit Requirement	REPORT	REPORT		*******	*******	*******				
BOD5	Sample Measurement			lb/day	*******			mg/L		03/07	24 HC
	Permit Requirement	1401	2102		*******	30	45				
TSS	Sample Measurement			lb/day	*******			mg/L		03/07	24 HC
	Permit Requirement	1401	2102		*******	30	45				
BOD_5 PERCENT REMOVAL	Sample Measurement	*******	*******		*******		*******	%		1/MONTH	CALC.
	Permit Requirement	*******	*******		*******	80%	*******				
TSS PERCENT REMOVAL	Sample Measurement	*******	*******		*******		*******	%		1/MONTH	CALC.
	Permit Requirement	*******	*******		*******	80%	*******				
FECAL COLIFORM BACTERIA	Sample Measurement	*******	*******	***	*******			#/100 ml		05/07	GRAB
	Permit Requirement	*******	*******		*******	200	400				
pH	Sample Measurement	*******	*******	***		*******		STD. UNITS		07/07	GRAB
	Permit Requirement	*******	*******		6.0	*******	9.0				
TOTAL RESIDUAL CHLORINE	Sample Measurement		*******	lb/day	*******			mg/L		05/07	GRAB
	Permit Requirement	2.21	*******		*******	0.05	0.10				
COPPER (TOTAL RECOVERABLE)	Sample Measurement		*******	lb/day	*******			ug/L		02/30	24 HC
	Permit Requirement	1.00	*******		*******	21.3	35				
ZINC (TOTAL RECOVERABLE)	Sample Measurement		*******	lb/day	*******			ug/L		02/30	24 HC
	Permit Requirement	4.13	*******		*******	88.4	177.4				
AMMONIA NH_3-N (as N)	Sample Measurement		*******	lb/day	*******			mg/L		01/07	24 HC
	Permit Requirement	1448	*******		*******	31	41				

Parameter		QUANTITY OR LOADING			QUALITY OR CONCENTRATION				No. of Exceed-ances	Frequency of Analysis	Sample Type
		Average	Maximum	Units	Minimum	Average	Maximum	Units			
SEASONAL LIMITS THAT APPLY ONLY JULY 1 TO October 31											
AMMONIA NH_3-N (as N)	Sample Measurement			lb/day	*******	*******	*******			03/07	24 HC
	Permit Requirement	922	REPORT		*******	*******	*******				
SEASONAL LIMITS THAT APPLY ONLY WHEN SKAGIT RIVER FLOW IS BELOW 6000 CFS FROM JULY 1 TO NOVEMBER 15 Report the number of days the flow is below 6000 cfs for the reporting period >>> _____											
AMMONIA NH_3-N (as N)	Sample Measurement	*******		lb/day	*******	*******	*******			03/07	24 HC
	Permit Requirement	*******	1188		*******	*******	*******				

NAME/TITLE PRINCIPAL EXECUTIVE OFFICER	I CERTIFY UNDER PENALTY OF LAW THAT THIS DOCUMENT AND ALL ATTACHMENTS WERE PREPARED UNDER MY DIRECTION OR SUPERVISION IN ACCORDANCE WITH A SYSTEM DESIGNED TO ASSURE THAT QUALIFIED PERSONNEL PROPERLY GATHER AND EVALUATE THE INFORMATION SUBMITTED. BASED ON MY INQUIRY OF THE PERSON OR PERSONS WHO MANAGE THE SYSTEM, OR THOSE PERSONS DIRECTLY RESPONSIBLE FOR GATHERING THE INFORMATION, THE INFORMATION SUBMITTED IS, TO THE BEST OF MY KNOWLEDGE AND BELIEF, TRUE, ACCURATE, AND COMPLETE. I AM AWARE THAT THERE ARE SIGNIFICANT PENALTIES FOR SUBMITTING FALSE INFORMATION, INCLUDING THE POSSIBILITY OF FINE AND IMPRISONMENT FOR KNOWING VIOLATIONS.		TELEPHONE	DATE
TYPED OR PRINTED		SIGNATURE OF PRINCIPAL EXECUTIVE OFFICER OR AUTHORIZED AGENT	( ) AREA CODE NUMBER	/ / YEAR MO DAY

COMMENT AND EXPLANATION OF ANY VIOLATIONS (Reference all attachments here)

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

MOUNT VERNON WASTEWATER TREATMENT PLANT
OUTFALL MODIFICATIONS
PERMITTING EVALUATION

The City of Mount Vernon (City) owns and operates the Mount Vernon Wastewater Treatment Plant (WWTP) that provides activated sludge secondary treatment and anaerobic sludge digestion. The WWTP discharges through an existing open-ended outfall located along the bank of the Skagit River within the City's jurisdictional boundary. The City is evaluating modifications to its existing discharge system to improve effluent mixing and compliance with water quality standards.

Based on experience of the City of Burlington Public Works when doing work associated with their Wastewater Treatment Plant, ***in-water work in the Skagit River will most likely be limited to July through mid-September*** through the Corps authorization or Hydraulic Project approval (Garrett 2000).

A permitting evaluation is presented below and is based on a new discharge pipe, open trench crossing of the dike, and extending the outfall to the thalweg of the Skagit River. It has also been assumed for the purposes of this evaluation that there are no wetland impacts.

U.S. Army Corps of Engineers Section 10/404

Authorization from the U.S. Army Corps of Engineers (Corps) will be required for maintenance of the existing outfall or construction of a new outfall. It is possible that the project would be authorized under Nationwide Permit 7 – Outfall Structures and Maintenance which specifically allows "construction of outfall structures and associated intake structures where the effluent from the outfall is authorized, conditionally authorized, or specifically exempted, or is otherwise in compliance with regulations issued under the National Pollution Discharge Elimination System program" (Corps 2000a).

Because of recent Endangered Species Act (ESA) listings, the Corps is now requiring a Biological Evaluation/Biological Assessment (BE/BA) for all projects requiring Corps approval Corps 2000b). This will trigger consultation with the National Marine Fisheries Service (NMFS) and the U.S. Fish and Wildlife Service (USFWS). It is also recommended that the Washington Department of Fish and Wildlife (WDFW) be contacted for their Priority Habitat Species Lists so that they can be incorporated into the BE/BA. Chinook salmon, bull trout and bald eagle are known to occur in the project vicinity and will most likely, after consultation with NMFS/USFWS/WDFW, be included in the BE/BA.

It is now estimated that with the large backlog of projects under review, it could take 18-24 months for an authorization to be issued by the Corps. A Joint Aquatic Resource Permit Application (JARPA) and supporting documentation (i.e., BE/BA, conceptual design drawings, etc.) should be submitted as soon as outfall design is nearly complete.

In most cases, the Corps will adopt the State Environmental Policy Act (SEPA) document to satisfy National Environmental Policy Act (NEPA) review. The SEPA determination will need to be made at the time of JARPA submittal.

Washington Department of Fish and Wildlife

A Hydraulic Project Approval (HPA) is required from the Washington Department of Fish and Wildlife (WDFW) for any work below the ordinary high water mark of the Skagit River. The JARPA can be used to apply for the HPA. Applications must include general plans for the overall project and include complete plans and specifications for the proper protection of fish.

WDFW would typically issue the HPA within 45 days of receiving the completed JARPA. However, they have recently determined that they may liable if they issue an HPA that would result in a take of an ESA-listed species. To address that liability, WDFW is proposing to develop a programmatic ESA compliance agreement for the Hydraulic Project Approval program (WDFW 2000). Until their agreement is in place, review of the JARPA and issuance of the HPA may take longer than 45 days.

Washington Department of Ecology

NPDES Permit: The discharge of pollutants into the state's surface waters is regulated through National Pollutant Discharge Elimination System (NPDES) permits. The Washington Department of Ecology (Ecology) issues these permits under authority delegated by the U.S. Environmental Protection Agency. Permits typically place limits on the quantity and concentration of pollutants that may be discharged. NPDES permits are required for wastewater discharges to surface waters. It is assumed that the existing NPDES permit for the outfall discharge will require modification or a new NPDES permit may be required. It could take between 180 days to one year for an individual permit depending on the complexity of the issues. Modification of an existing permit may take less time

401 Water Quality Certification: An individual 401 Water Quality Certification may be required from Ecology. Application can be made with the JARPA and review can be concurrent with the Corps review. The 401 Water Quality Certification must be issued within one year of submittal or will be considered waived.

Washington Department of Natural Resources

Any project that is located on state-owned aquatic lands will require authorization from the Washington Department of Natural Resources (WDNR). For the purposes of this evaluation, it has been assumed that the outfall is located on state-owned lands. Application for an Aquatic Lease can take up to one year to be issued.

City of Mount Vernon

State Environmental Policy Act (SEPA): As a non-federal governmental agency, the City of Mount Vernon will be the lead agency for SEPA. It is anticipated that an Environmental Checklist will be prepared and a Determination of Nonsignificance (DNS) issued. Because of the need to submit with the Corps Permit, the SEPA review needs to be completed very early in the permitting process.

Shoreline Substantial Development Permit: All activities within 200 feet of the Skagit River (including in-water work) are subject to Shoreline Substantial Development Permit review. The City of Mount Vernon (City) requires a pre-application conference to occur prior to application submittal (other applications can be included in this review). Once the application is submitted, the City will review the application and prepare a Notice of Technically Complete Application. A Shoreline Permit is subject to a public hearing before a Hearing Examiner and must be scheduled within 60 days of the notice of complete application. The City will issue a Notice of Decision on the project, typically 120 calendar days from the Notice of Technically Complete Application (Mt. Vernon 2000).

Dike Setback Variance: Because this project will involve construction within the dike setback restriction area along the Skagit River, a variance from the provisions of the Mount Vernon Municipal Code, Section 15.36.270, will be required. The variance will be approved by a hearing examiner and will run concurrent with the Shoreline Substantial Development Permit process. The hearing examiner shall consider all technical evaluations, relevant factors, and criteria presented in 15.36.150(D) 1 through 11.

Critical Areas/Floodplain Review: Specific projects impacts have not been determined, however, critical areas and floodplain review are two areas that are likely for this project. Although the project will be located in a floodplain, it is not anticipated that the project will impact the floodplain. A review of applicable approvals/permits will be determined once impacts are known.

Fill and Grading Permit: A fill and grading permit will be required for the open trenching required for the 800 feet of discharge pipe. The application can be submitted after the land use permitting process (shorelines) is complete.

Dike District #3 Approval

The installation of the new discharge pipe will require an open-cut crossing of the dike along the Skagit River. The dikes in the project area are subject to review by Dike District #3 (Eisses 2000). It is anticipated that the open-cut crossing of the dikes will be allowed by the Dike District. Mount Vernon will need to submit final plans and specifications for review and approval prior to work associated with the dikes. The Dike District meets once a month, but can meet more frequently if needed (Smith 2000).

References

Corps 2000a. U.S. Army Corps of Engineers, Seattle District. Special Public Notice – Final Regional Conditions, 401 Water Quality Certification Conditions, Coastal Zone Management consistency Responses for Nationwide Permits for the Seattle district corps of Engineers for the State of Washington. June 16, 2000.

Corps 2000b. U. S. Army Corps of Engineers, Seattle District. Special Public Notice – Corps of Engineers Regulatory Program and the Endangered Species act. April 11, 2000.

Eisses, Dan. 2000. Personal communication with Dan Eissess, Mount Vernon Public Works. November 15, 2000.

Garrett, Rod. 2000. Personal communication with Rod Garrett, City of Burlington Public Works Director. November 15, 2000.

Mt. Vernon 2000. City of Mount Vernon, Community and Economic Development Department Land Use Permit Processing Procedures. www.mount-vernon.wa.us/ced/landuse.htm. September 25, 2000.

Smith, Richard. 2000. Personal communication with Richard Smith, Dike District #3. November 20, 2000.

WDFW 2000. Washington Department of Fish and Wildlife. ESA Compliance Report. www.wa.gov/wdfw/hab/hpa/hpahcp.htm.

Appendix J
Mount Vernon WWTP
UV Transmittance Test Results

Trojan

TROJAN TECHNOLOGIES INC.
3020 Gore Road
London, Ontario
N5V 4T7 CANADA
Telephone: **(519) 457-3400**
Facsimile: **(519) 457-3030**

WATER ANALYSIS REPORT

To: Victoria Falvo

Rep: Wm. H. Reilly & Co.

Engineer: HDR Engineering

Project Name: **Mount Vernon, WA**

Sample #: S01-2878 to S01-2881

Sample Source: Mount Vernon Wastewater Utility

Process: Conventional Activated Sludge

Date sample taken: June 04, 2001

Date sample analysed: June 06, 2001

Disinfection Limits: 200 Fecal Coliform/100mL

Parameters Analyzed: UV Transmittance-whole sample, UV Transmittance – filtered & TSS

Sample Treatment: No Additives

SAMPLE NO.	SAMPLE DESCRIPTION	%T	%T FILTERED	TSS (PPM)	MEAN PARTICLE SIZE (MICRONS)	%PARTICLES >31MICRONS
S01-2878	Primary Effluent, flow @ 3.31 June 04, 2001	14	26	81	-	-
S01-2879	Primary Effluent, flow @ 3.31, PSA June 04, 2001	-	-	-	23.4	22.5
S01-2880	Final Effluent, flow @ 3.31 June 04, 2001	62	64	3	-	-
S01-2881	Final Effluent, flow @ 3.31, PSA June 04, 2001	-	-	-	31.6	37.3

Comments:

Certified by J. Faber,
Team Leader, Analytical Services

APPENDIX K
MOUNT VERNON WWTP MIXING ZONE STUDY

City of Mount Vernon

Mount Vernon WWTP
Mixing Zone Study

Prepared for:

City of Mount Vernon
1401 Britt Road
Mount Vernon, Washington 98273

Prepared by:

Cosmopolitan Engineering Group
117 South 8th Street
Tacoma, Washington 98402

February 2000
MTV002

General Instructions

f form has been partially completed by preprinting, disregard instructions directed at entry of that information already preprinted.

Enter *"Permittee Name/Mailing Address* (and facility name/location, if different)," *"Permit Number,"* and *"Discharge Number"* where indicated. (A separate form is required for each discharge.)

3. Enter dates beginning and ending *"Monitoring Period"* covered by form where indicated.

4. Enter each *"Parameter"* as specified in monitoring requirements of permit.

5. Enter *"Sample Measurement"* data for each parameter under *"Quantity"* and *"Quality"* in units specified in permit. *"Average"* is normally arithmetic average (geometric average for bacterial parameters) of all sample measurements for each parameter obtained during *"Monitoring Period"; "Maximum"* and *"Minimum"* are normally extreme high and low measurements obtained during *"Monitoring Period."* **(Note to municipals with secondary treatment requirement:** Enter 30-day average of sample measurements under *"Average,"* and enter maximum 7-day average of sample measurements obtained during monitoring period under *"Maximum."*)

6. Enter *"Permit Requirement"* for each parameter under *"Quantity"* and *"Quality"* as specified in permit.

7. Under *"No. of Exceedances"* enter number of sample measurements during monitoring period that exceed maximum (and/or minimum or 7-day average and monthly average as appropriate) permit requirement for each parameter. If none, enter *"O."*

8. Enter *"Frequency of Analysis"* both as *"Sample Measurement"* (actual frequency of sampling and analysis used during monitoring period) and as *"Permit Requirement"* specified in permit. (e.g., Enter *"Cont,"* for continuous monitoring, *"1/7"* for one day per week, *"1/30"* for one day per month, *"1/90"* for one day per quarter, etc.)

9. Enter *"Sample Type"* both as *"Sample Measurement"* (actual sample type used during monitoring period) and as *"Permit Requirement,"* (e.g., Enter *"Grab"* for individual sample, *"24HC"* for 24-hour composite, *"N/A"* for continuous monitoring, etc.)

10. Where violations of permit requirements are reported, attach a brief explanation to describe cause and corrective actions taken, and reference each violation by date.

If "no discharge" occurs during monitoring period, check the *"No Discharge"* box in the upper right-hand corner of page 1.

12. Enter *"Name/Title of Principal Executive Officer"* with *"Signature of Principal Executive Officer of Authorized Agent," "Telephone Number,"* and *"Date"* at bottom of form.

13. Mail signed Report to Office(s) by date(s) specified in permit. Retain copy for your records.

14. More detailed instructions for use of this *Discharge Monitoring Report (DMR)* form may be obtained from Office(s) specified in permit.

Legal Notice

This report is required by law (33 U.S.C. 1318; 40 C.F.R. 125.27). Failure to report or failure to report truthfully can result in civil penalties not to exceed $10,000 per day of violation; or in criminal penalties not to exceed $25,000 per day of violation, or by imprisonment for not more than one year, or by both.

APPENDIX H

ENVISION MODEL DATA SUMMARY SHEETS FOR THE CITY OF MOUNT VERNON WASTEWATER TREATMENT PLANT

UnitProc	Parameter	Condition	Range	Limit	Unit	Comment	Source
Primary Clarifiers	OFR	MM15	800-1200	1,000	gpd/sf		DOE Standard
Primary Clarifiers	OFR	PH15	2000-3000	2,500	gpd/sf		DOE Standard
Primary Clarifiers	HRT	MM15	2.5	2.5	hr		DOE Standard
Aeration Basins	MLSS	MM15		2,500	mg/L	assumes existing 11 SWD clarifier is not used	Stress Testing
Aeration Basins	MLSS	MD15		2,700	mg/L	assumes existing 11 SWD clarifier is not used	Stress Testing
Aeration System	OUR	MM15		45	mg/L/h	Based on fine bubble diffusers	HDR Standard
Aeration System	OUR	MD15		50	mg/L/h	Based on fine bubble diffusers	HDR Standard
Aeration System	OUR	PH15		75	mg/L/h	Based on fine bubble diffusers	HDR Standard
Blower	SCFM	MD15		12,800	scfm	4@4,100 scfm, assume 1 out of service	Installed firm capacity
Blower	SCFM	PH15		12,300	scfm	4@4,100 scfm, assume 1 out of service	Installed firm capacity
Secondary Clarifiers	HRT	MD15		2	hr	With RAS - sustained load	HDR Standard
Secondary Clarifiers	HRT	PH15		2	hr	With RAS - sustained load	HDR Standard
Secondary Clarifiers	OFR	PH15		900	gpd/sf	Parameter lowered due to clarifier concern	DOE Peak Criteria Rate is 1200
Secondary Clarifiers	SLR	MM15		25	lb/d/sf	assumes existing 11 SWD clarifier is not used	DOE Standard
Secondary Clarifiers	SLR	MD15		40	lb/d/sf	assumes existing 11 SWD clarifier is not used	DOE Standard
Secondary Clarifiers	SLR	PH15		40	lb/d/sf	assumes existing 11 SWD clarifier is not used	DOE Standard
DAF Thickeners	SLR	MM15	1-2.5	2.5	lb/d/sf	Design criteria sheet says 1 pph w/o polymer, 2 pph	DOE Standard for DAF with polymer
Anaerobic Digester	HRT	MM15	15	15	d		EPA Sludge regulations
Anaerobic Digester	SLR	MM15	120-160	140	lb VSS/1000 cf/d		WEF MOP8
Gravity Thickener	OFR	MM15	600-800	700	gpd/sf		DOE Standard

UnitProc	Parameter	Condition	Range	Limit	Unit	Comment	Source
Primary Clarifiers	OFR	MM15	800-1200	1,000	gpd/sf		DOE Standard
Primary Clarifiers	OFR	PH15	2000-3000	2,500	gpd/sf		DOE Standard
Primary Clarifiers	HRT	MM15	2.5	2.5	hr		DOE Standard
Aeration Basins	MLSS	MM15		2,500	mg/L	assumes existing 11 SWD clarifier is not used	Stress Testing
Aeration Basins	MLSS	MD15		2,700	mg/L	assumes existing 11 SWD clarifier is not used	Stress Testing
Aeration System	OUR	MM15		45	mg/L/h	Based on fine bubble diffusers	HDR Standard
Aeration System	OUR	MD15		50	mg/L/h	Based on fine bubble diffusers	HDR Standard
Aeration System	OUR	PH15		75	mg/L/h	Based on fine bubble diffusers	HDR Standard
Blower	SCFM	MD15		12,300	scfm	4@4,100 scfm, assume 1 out of service	Installed firm capacity
Blower	SCFM	PH15		12,300	scfm	4@4,100 scfm, assume 1 out of service	Installed firm capacity
Secondary Clarifiers	HRT	MD15		2	hr	With RAS - sustained load	HDR Standard
Secondary Clarifiers	HRT	PH15		2	hr	With RAS - sustained load	HDR Standard
Secondary Clarifiers	OFR	PH15		900	gpd/sf	Parameter lowered due to clarifier concern	DOE Peak Criteria Rate is 1200
Secondary Clarifiers	SLR	MM15		25	lb/d/sf	assumes existing 11 SWD clarifier is not used	DOE Standard
Secondary Clarifiers	SLR	MD15		40	lb/d/sf	assumes existing 11 SWD clarifier is not used	DOE Standard
Secondary Clarifiers	SLR	PH15		40	lb/d/sf	assumes existing 11 SWD clarifier is not used	DOE Standard
DAF Thickeners	SLR	MM15	1-2.5	2.5	lb/d/sf	Design criteria sheet says 1 pph w/o polymer, 2 pph	DOE Standard for DAF with polym
Anaerobic Digester	HRT	MM15	15	15	d		EPA Sludge regulations
Anaerobic Digester	SLR	MM15	120-160	140	lb VSS/1000 cf/d		WEF MOP8
Gravity Thickener	OFR	MM15	600-800	700	gpd/sf		DOE Standard

UnitProc	Parameter	Condition	Range	Limit	Unit	Comment
Primary Clarifiers	OFR	MM15	800-1200	1,000	gpd/sf	
Primary Clarifiers	OFR	PH15	2000-3000	2,500	gpd/sf	
Primary Clarifiers	HRT	MM15	2.5	2.5	hr	
Aeration Basins	MLSS	MM15		2,500	mg/L	assumes existing 11 SWD clarifier is not used
Aeration Basins	MLSS	MD15		2,700	mg/L	assumes existing 11 SWD clarifier is not used
Aeration System	OUR	MM15		45	mg/L/h	Based on fine bubble diffusers
Aeration System	OUR	MD15		50	mg/L/h	Based on fine bubble diffusers
Aeration System	OUR	PH15		75	mg/L/h	Based on fine bubble diffusers
Blower	SCFM	MD15		12,300	scfm	4@4,100 scfm, assume 1 out of service
Blower	SCFM	PH15		12,300	scfm	4@4,100 scfm, assume 1 out of service
Secondary Clarifiers	HRT	MD15		2	hr	With RAS - sustained load
Secondary Clarifiers	HRT	PH15		2	hr	With RAS - sustained load
Secondary Clarifiers	OFR	PH15		900	gpd/sf	Parameter lowered due to clarifier concern
Secondary Clarifiers	SLR	MM15		25	lb/d/sf	assumes existing 11 SWD clarifier is not used
Secondary Clarifiers	SLR	MD15		40	lb/d/sf	assumes existing 11 SWD clarifier is not used
Secondary Clarifiers	SLR	PH15		40	lb/d/sf	assumes existing 11 SWD clarifier is not used
DAF Thickeners	SLR	MM15	1-2.5	2.5	lb/d/sf	Design criteria sheet says 1 pph w/o polymer, 2 pph
Anaerobic Digester	HRT	MM15	15	15	d	
Anaerobic Digester	SLR	MM15	120-160	140	lb VSS/1000 cf/d	
Gravity Thickener	OFR	MM15	600-800	700	gpd/sf	

UnitProc	Parameter	Condition	Range	Limit	Unit	Comment	Source
Primary Clarifiers	OFR	MM15	800-1200	1,000	gpd/sf		DOE Standard
Primary Clarifiers	OFR	PH15	2000-3000	2,500	gpd/sf		DOE Standard
Primary Clarifiers	HRT	MM15	2.5	2.5	hr		DOE Standard
Aeration Basins	MLSS	MM15		2,500	mg/L	assumes existing 11 SWD clarifier is not used	Stress Testing
Aeration Basins	MLSS	MD15		2,700	mg/L	assumes existing 11 SWD clarifier is not used	Stress Testing
Aeration System	OUR	MM15		45	mg/L/h	Based on fine bubble diffusers	HDR Standard
Aeration System	OUR	MD15		50	mg/L/h	Based on fine bubble diffusers	HDR Standard
Aeration System	OUR	PH15		75	mg/L/h	Based on fine bubble diffusers	HDR Standard
Blower	SCFM	MD15		12,300	scfm	4@4,100 scfm, assume 1 out of service	Installed firm capacity
Blower	SCFM	PH15		12,300	scfm	4@4,100 scfm, assume 1 out of service	Installed firm capacity
Secondary Clarifiers	HRT	MD15		2	hr	With RAS - sustained load	HDR Standard
Secondary Clarifiers	HRT	PH15		2	hr	With RAS - sustained load	HDR Standard
Secondary Clarifiers	OFR	PH15		900	gpd/sf	Parameter lowered due to clarifier concern	DOE Peak Criteria Rate is 1200
Secondary Clarifiers	SLR	MM15		25	lb/d/sf	assumes existing 11 SWD clarifier is not used	DOE Standard
Secondary Clarifiers	SLR	MD15		40	lb/d/sf	assumes existing 11 SWD clarifier is not used	DOE Standard
Secondary Clarifiers	SLR	PH15		40	lb/d/sf	assumes existing 11 SWD clarifier is not used	DOE Standard
DAF Thickeners	SLR	MM15	1-2.5	2.5	lb/d/sf	Design criteria sheet says 1 pph w/o polymer, 2 pph	DOE Standard for DAF with polymer
Anaerobic Digester	HRT	MM15	15	15	d		EPA Sludge regulations
Anaerobic Digester	SLR	MM15	120-160	140	lb VSS/1000 cf/d		WEF MOP8
Gravity Thickener	OFR	MM15	600-800	700	gpd/sf		DOE Standard

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SECTION 1:
INTRODUCTION

1.1 BACKGROUND

The City of Mount Vernon owns and operates the Mount Vernon Wastewater Treatment Plant (WWTP) under NPDES permit No. WA-002407-4. The treatment plant provides activated sludge secondary treatment and anaerobic sludge digestion. The plant was upgraded in 1989 to a design population of 28,500. A Wastewater Comprehensive Plan was completed in 1998, with a projected sewered population of 43,560 in 2015, plus additional commercial and industrial sources.

1.2 SKAGIT RIVER TMDL

Ecology completed the *Lower Skagit River Total Maximum Daily Load Water Quality Study* (Pickett, 1997), which included a model of dissolved oxygen in the Skagit River from RM 24.6 (near Sedro Woolley) to RM 0.0 (Puget Sound). The report recommended wasteload allocations of BOD and ammonia for each of the municipal WWTPs in the lower river. The model has been revised based on subsequent field data collected in 1998, and the recommended wasteload allocations have been modified.

1.3 NPDES PERMIT RENEWAL

The Department of Ecology is expected to renew Mount Vernon's NPDES permit in 2000. According to Ecology, the permit will include effluent limits for ammonia and BOD derived from the revised Skagit River TMDL model (personal communication with Jerry Shervey, 1999). The permit may also include effluent limits based on compliance with water quality standards within authorized mixing zones.

1.4 PURPOSE

This study assesses effluent mixing for the existing open-ended outfall located along the bank of the Skagit River. The study also evaluates the need for and benefit of an outfall extension and/or addition of a diffuser to improve effluent mixing and compliance with water quality standards.

The purpose of this study is to establish the following for several outfall improvement alternatives:

- Water quality-based effluent limits for the next NPDES permit
- Projected effluent limits for 2015 effluent flows

This study satisfies the *Engineering Report* requirements for outfalls specified in WAC 173-280-060(d), (e), and (l). The results of this study will be incorporated into *the 2000 Comprehensive Sewer Plan Update* and *Wastewater Facility Plan* currently being developed by HDR Engineering.

1.5 SEASONAL SCOPE

The analysis of outfall alternatives, effluent mixing and water quality impacts is limited to the low flow period of the Skagit River, which occurs between July and October. This is the period when the river is most vulnerable to water quality impacts from the discharge of toxicants and oxygen demanding wastes from the WWTP. It is also the period when the TMDL wasteload allocations for BOD and ammonia will apply.

The City of Mount Vernon will be constructing improvements in their collection system to reduce CSO discharges. These improvements will increase peak effluent flows from the treatment plant during the wet season. These improvements may require construction of a larger outfall or a second outfall. The need for a second outfall and the associated wet weather water quality impacts are not evaluated in this report. Because peak flows are not established, engineering design criteria for outfall improvements are also not evaluated in the report. These design issues would be addressed during predesign if an outfall improvement is implemented.

SECTION 2:
DESIGN CRITERIA

2.1 EFFLUENT FLOWS

Existing and projected effluent flows for the critical dry weather period are shown in Table 1. Maximum monthly average flows are used for assessing chronic toxicity, and maximum daily flows are used for acute toxicity (Ecology, 1992). The 1999 effluent flows are based on a review of the last five years' Daily Monitoring Reports for August through October. This is the record that Ecology will use to establish water quality-based effluent limits in the next NPDES permit. The 2015 flows are conservative estimates based on the *1998 Comprehensive Plan*. The 2015 flows will be re-evaluated or verified by HDR Engineering in the *2000 Comprehensive Sewer Plan Update*.

Table 1 Design Effluent Flow Rates for July through October

Planning Year	Maximum Month (mgd)	Maximum Day (mgd)
1999	3.51	6.38
2015	8.13	12.86

2.2 WATER QUALITY STANDARDS

The Skagit River is rated Class A fresh water according to the Washington State Water Quality Standards WAC 173-201A. The toxic substances that typically appear in municipal wastewater effluent include chlorine, ammonia, and metals. The numerical standards for these parameters are provided in Table 2.

The water quality standards for ammonia are based on ambient pH and temperature. The ammonia standards shown in Table 2 are the 5th percentile of the water quality criteria calculated for July through October ambient monitoring data collected by Ecology near Mount Vernon from 1991 through 1997. Metals criteria are based on an ambient hardness of 25 mg/L as $CaCO_3$. Ambient water quality data and criteria worksheets are provided in Appendix A.

Table 2 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

2.3 MIXING ZONES

The Mount Vernon NPDES permit authorizes acute and chronic mixing zones, which are described in WAC 173-201A-100. The allowed mixing zone for chronic water quality criteria:

- 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
- shall not occupy greater than 25 percent of the river width

The allowed mixing zone for acute water quality criteria:

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

SECTION 3:
EXISTING OUTFALL

3.1 DESCRIPTION

The Mount Vernon WWTP discharges secondary-treated effluent to the Skagit River at RM 10.7. The existing outfall for the Mount Vernon WWTP consists of a 24-inch diameter, Class 5 ductile iron restrained joint pipe. The pipe is open-ended, discharging at the river's edge at a depth of 8 feet. The outfall profile for the river section is shown in Figure 1.

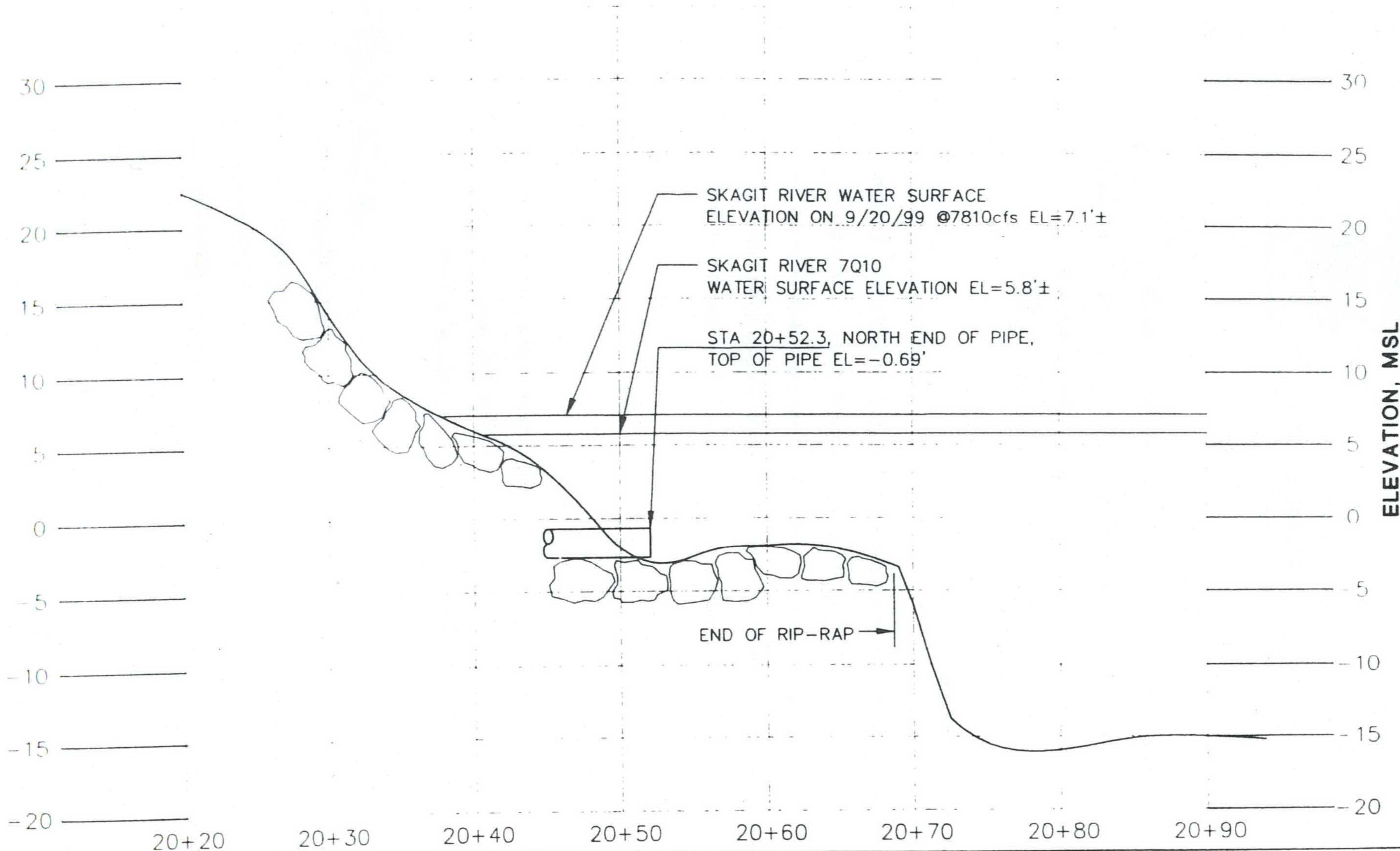
3.2 OUTFALL INSPECTION

The outfall was visually inspected by divers during a field study conducted on September 20, 1999. The outfall was found intact and generally as shown in Figure 1. There are no joints exposed, and the terminus has a plain end. The outfall is well armored along the slope and at the terminus. However, there is an abrupt 8-foot vertical drop-off located approximately 15 feet offshore of the terminus, which is not shown on the drawings. The divers report is furnished in Appendix B.

3.3 DISCHARGE EDDY

The outfall is located within a small depression in the riverbank in plan view. This depression creates an eddy (or gyre) that is visibly trapping effluent near the shoreline. Rhodamine WT dye was injected into the effluent for approximately 30 minutes on October 5, 1999, to confirm current effluent concentrations along the shoreline.

The dye was clearly visible in the eddy formed in the riverbank depression. Photos are provided in Figure 2. Approximate locations of the outfall and acute mixing zone boundaries have been superimposed on the photos. Effluent was visibly trapped within the eddy. The mixing of effluent with ambient water occurred at the offshore boundary of the eddy, where there was strong turbulence between the circulating eddy and the ambient flow.



FILE: MTV002 JOB: MTV002F1

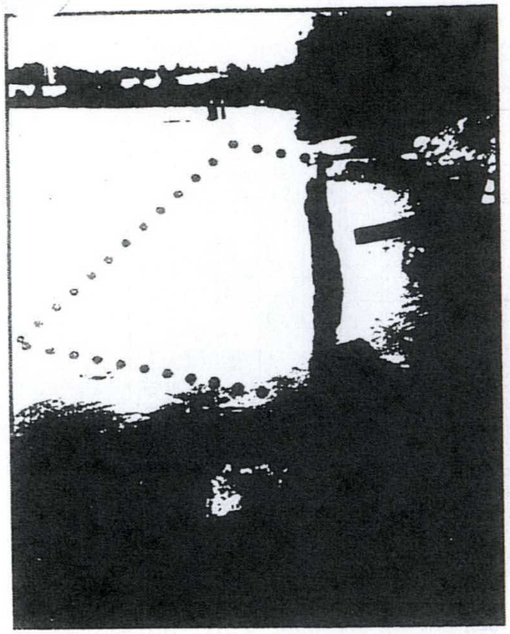
ENGINEERING GROUP

Civil, Environmental and Recreational Consulting

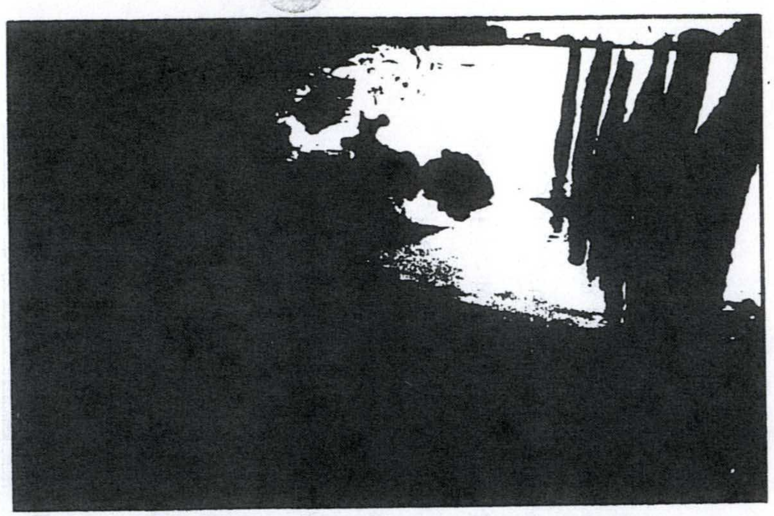
SCALES:
1" = 10' HOR
1" = 10' VER

Figure 1 Existing Outfall Profile

Scale 10 = 1'-0"
P:\MTV002\Graphics\File MTV002F1.DWG
Date 2/10/2000 12:59 P.M.

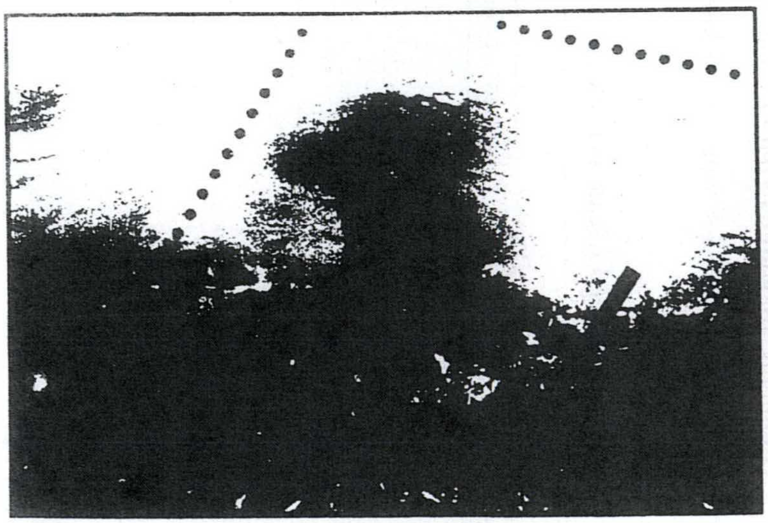


50 feet downstream looking upstream

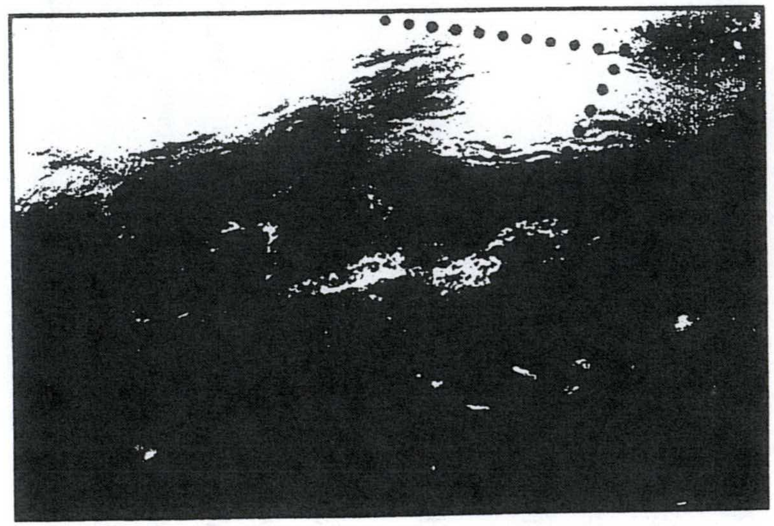


50 feet downstream looking downstream

Legend:
Outfall
Acute Mixing Zone



Outfall to downstream AMZ boundary



Outfall to upstream AMZ boundary

Figure 2 Effluent Plume Photos
October 5, 1999

SECTION 4:
FIELD STUDIES

4.1 VELOCITY AND DEPTH

Velocity and depth measurements were obtained on September 20, 1999. Depth measurements were a combination of direct soundings and acoustic soundings taken over the entire cross-section. Cross-section data are plotted in Figure 3.

4.1.1 Low Tide Measurements

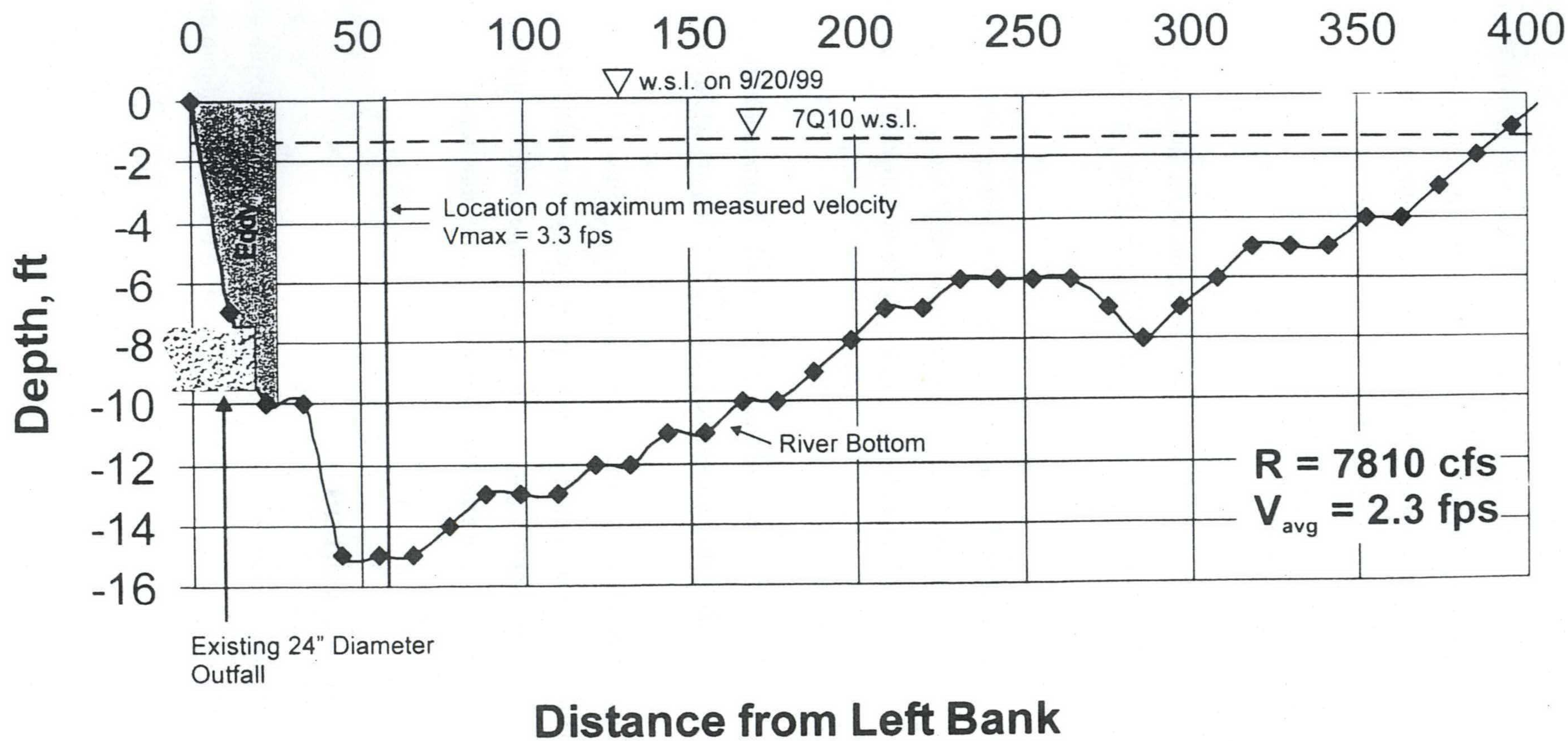
Velocity measurements were obtained at maximum 10-foot intervals for the 80 feet nearest the shoreline with a Swoffer current meter with digital readout. Readings were obtained at 20, 60 and 80 percent of the depth. Readings were taken during low tide between 13:30 and 14:30 on September 20, when the Skagit River discharge measured at USGS Station 12200500 near Mount Vernon was 7,810 cfs. Readings were repeated at 17:30 when there was some tidal influence; however, river discharge had increased to approximately 9,800 cfs at that point.

Velocity measurements are summarized in Table 3. The maximum depth-averaged velocity was approximately 3.3 fps, and the average was calculated from the cross-section as 2.3 fps.

Table 3 Current Speed Measurements – September 20, 1999

Distance from Left Bank (ft)	Depth (ft)	Velocity (fps)			
		20% Depth	60% Depth	80% Depth	Average
5	3.3	–	1.36	–	1.4
10	7	1.17	1.43	1.21	1.3
18	12	1.73	1.96	1.40	1.7
28	11	2.33	2.28	2.31	2.3
38	15	2.58	2.65	–	2.6
48	15	2.72	3.15	–	2.9
58	14	3.10	3.04	2.70	3.0
78	13	3.54	3.40	2.98	3.3

Figure 3
Skagit River Cross-Section 9/20/99



Velocity measurements were not obtained within the eddy surrounding the outfall because the direction of the currents within the eddy was very unsteady. Based on visual observations, the currents inside the eddy were vigorous, but produced little negligible net downstream movement of the effluent or exchange with ambient waters.

4.1.2 High Tide Measurements

The Skagit River at the Mount Vernon WWTP has a very slight tidal influence at high tides. The high tide raises the water surface slightly, but there is no flow stoppage or current reversal, and no salt wedge is present.

The field studies were conducted on September 20, 1999, because there was a high tide in Skagit Bay at 3:30 p.m. The highest tide level was observed at the outfall site, 10.7 miles from Skagit Bay, at approximately 5:30 p.m. Water surface elevation rose an estimated 1.2 feet from the earlier low tide measurements.

River discharge at Mount Vernon typically increases in late afternoon, including this field study. River discharge at Station 12200500 increased to 9,290 cfs at 5:30 p.m. on September 20, 1999.

Velocity measurements were slightly higher than those observed at low tide. Peak velocity was approximately 3.4 feet per second, and the cross-sectional average is estimated at 2.5 feet per second. Since the velocity and depth were greater during the late afternoon tides, the low tide depth and velocity measurements are selected as the critical conditions for dilution modeling and setting water quality-based limits.

4.2 EFFLUENT TRACER STUDIES

Effluent concentrations were measured on October 5, 1999, within the visible plume trapped in the previously discussed eddy. Rhodamine WT fluorescent dye was injected into the effluent for approximately 30 minutes at a constant rate, and plume samples were taken with a Niskin bottle sampler. Fluorescence of effluent and river samples was measured in a laboratory with a Turner Model 10 fluorometer.

Temperature was also used as a tracer. Temperature was measured in effluent and in the river with a SeaBird Model SBE-16. The temperature data are shown in Figure 4 and Appendix C.

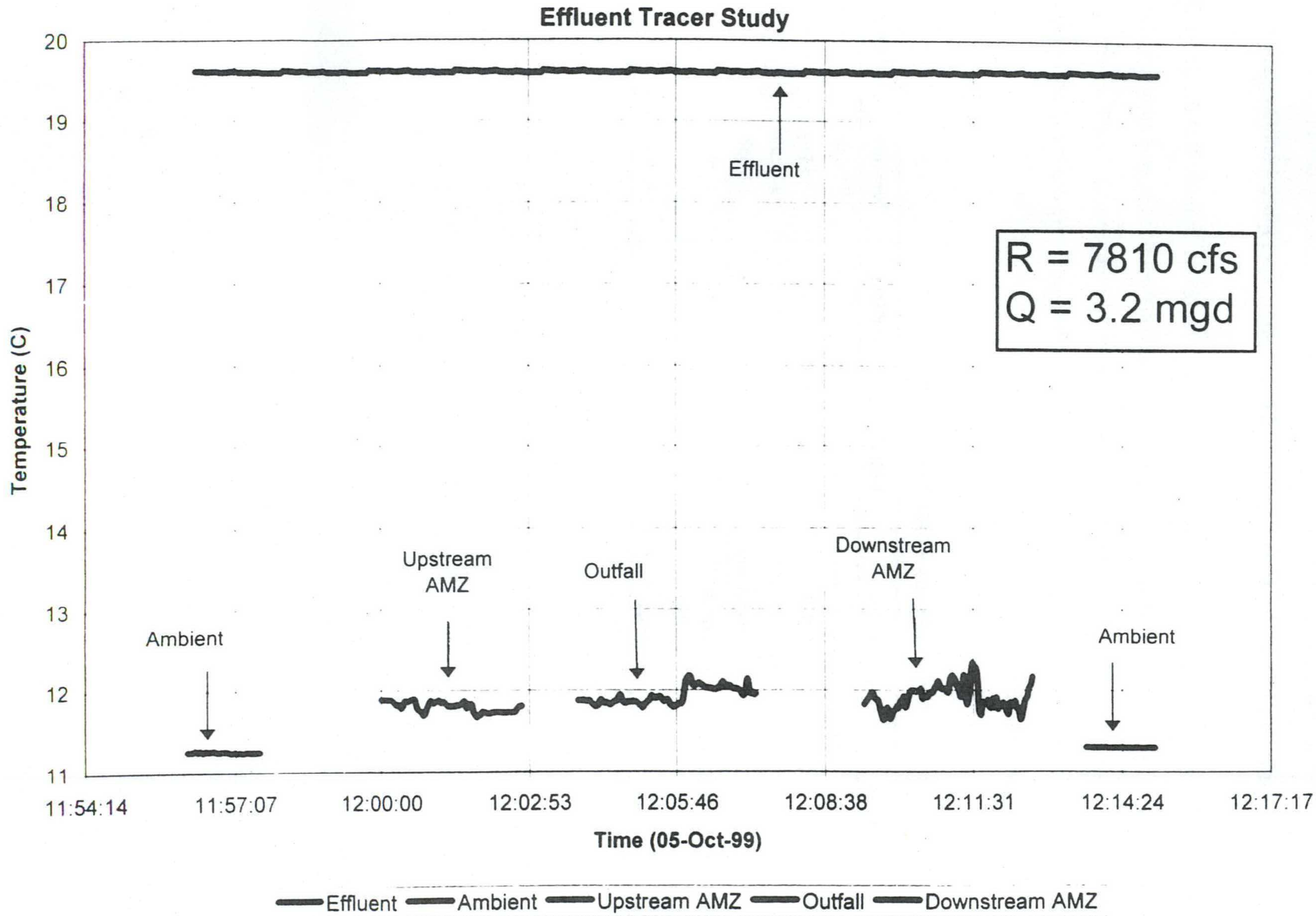
The results of the tracer studies are provided in Table 4. The results indicate that the dilution factor at both the upstream and downstream acute mixing zone boundaries was approximately 13:1 at the time of the study. The effluent flow rate during the study was 3.2 mgd and the river discharge was 7,810 cfs measured at USGS Station 12200500, the same discharge as during the September 20 field study.

Table 4 Tracer Study Results – October 5, 1999

Tracer	Location	Effluent	Ambient	Measured	Dilution
Rhodamine WT Dye	Upstream AMZ	117 ppb	0	9.9 ppb	11.8
	Outfall			24.4 ppb	4.8
	Downstream AMZ			9.1 ppb	12.9
	200'± Downstream			1.8 ppb	65
Temperature	Upstream AMZ	19.6°C	11.3°C	11.9°C	13
	Outfall			12.1°C	10
	Downstream AMZ			11.9°C	13

AMZ = Acute Mixing Zone boundary

Figure 4
Temperature Tracer Study Data



SECTION 5:
MIXING ZONE MODELING

5.1 OUTFALL ALTERNATIVES

The objective of modifying the existing outfall would be to inject effluent where it would not be trapped in the shoreline eddies. The current outfall is located at an outside bend in the river, where historical cross-sections and aerial photos furnished by Skagit County and the Corps of Engineers reveal that the thalweg has been relatively stable along the left bank. The proposed project would extend the outfall approximately 40 feet from its present location to a terminus near the thalweg. The profile is shown schematically in Figure 5.

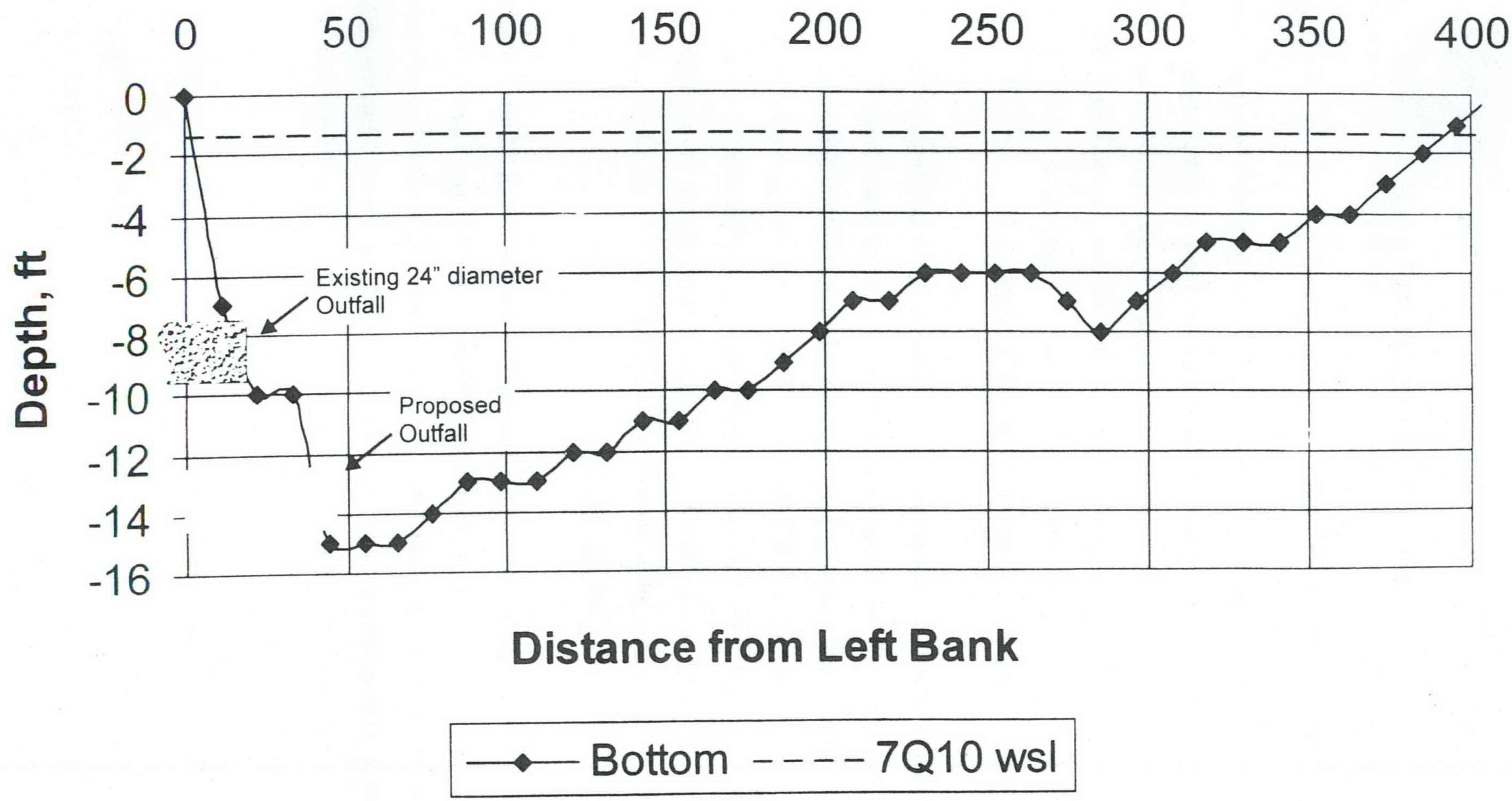
The outfall could include a multi-port diffuser. However, the maximum acute and chronic dilution factors allowed in the mixing zone regulations are limited to a fixed percent of the ambient river flow. A diffuser may actually achieve a higher dilution factor at the mixing zone boundary than allowed by the WAC. Base on preliminary modeling, it is anticipated that the maximum allowable dilution could be achieved with an open-ended outfall located at the thalweg with an upward nozzle trajectory. Diffusers are undesirable from a reliability standpoint because of the potential for debris damage and plugging ports. Therefore, an open-ended discharge is the preferred discharge alternative.

The required hydraulic design criteria for the outfall extension are not currently available due to the ongoing facility planning for CSO control. Therefore, this report does not include a detailed engineering evaluation of the outfall alternatives. Pipe material, diameter, alignment and profile, anchoring and armoring would be considered during a predesign phase if this option is selected and would be coordinated with the CSO improvements. The need for a diffuser would also be confirmed during this phase.

5.2 DILUTION MODEL SELECTION

The NPDES *Permit Writer's Manual* (Ecology, 1992) and the Orange Book (Ecology, 1998) describe several models appropriate for the river environment. The models PLUMES, CORMIX and RIVPLUM5 have been considered for this application.

Figure 5
Proposed Outfall Extension Schematic



PLUMES was used to evaluate the nearfield hydrodynamics of the discharge plume. This model provides a preliminary recommendation that a new outfall at the thalweg should be angled up at 45 degrees in order to disperse the effluent vertically. Model output comparing horizontal discharge to 45 degrees is provided in Appendix D. However, the PLUMES model was not used for dilution prediction because the model does not adequately consider the boundary conditions.

CORMIX was not selected for this application because it would not accurately simulate the existing discharge located in the eddy. It has little flexibility to adjust parameters for calibration to observed condition, and is not convenient to use because of the input file format.

RIVPLUM5 was chosen for this application for several reasons. It is the preferred model in rivers for point source discharges if the effluent plume is vertically mixed, which was demonstrated using the PLUMES model if the port is angled upward. RIVPLUM5 is conservative in that the initial velocity and buoyancy of the discharge are neglected. In addition, it is a spreadsheet model that can be run economically for multiple simulations. Finally, there are several options available to allow calibration to plume concentrations observed in the October 5 tracer studies for the existing outfall.

5.3 CRITICAL AMBIENT CONDITIONS

The *Permit Writer's Manual* stipulates that mixing zone dilution factors be calculated at the 7-day low flow with a 10-year recurrence interval (7Q10) derived from river discharge data. The 7Q10 discharge for the USGS Station Skagit River near Mount Vernon has been established at 5,030 cfs (Cosmopolitan Engineering Group, 1999).

<u>Thalweg</u>. Current speed and water depth are adjusted from the 7,810 cfs discharge observed September 20, 1999, to the 7Q10 discharge by the following method:

- The critical hydraulic condition is the non-tidally influenced (low tide) period as described in Section 4.1.
- 7Q10 water depth is lower than observed by approximately 1.3 feet, based on the rating curve for the USGS Station Skagit River near Mount Vernon.

- The average current speed of 2.3 fps observed on September 20 is reduced to 1.8 fps for 7Q10 conditions based on the reduced cross-sectional area.

The projected 7Q10 water surface is shown on Figures 3 and 5.

<u>Eddy</u>. Currents within the eddy are vigorous, but do not produce corresponding dilution factors due to recirculation or trapping of effluent, known as reflux. Dilution of the effluent plume occurs through turbulent entrainment along the outer edge of the eddy. Therefore, the flux of ambient water available to dilute the effluent is less than suggested by the speed of the currents within the eddy. The effective current for modeling the existing discharge is established empirically by adjusting current speed in RIVPLUM5 until the dilution predicted at the acute mixing zone boundary matches that observed in the tracer studies. The result is an artificial current speed of 0.1 fps for the October 5 tracer study, adjusted to 0.9 fps for the 7Q10 condition. The RIVPLUM5 model output for the calibration case is provided in Table 5. The shear velocity and transverse mixing coefficient are also artificial, reflecting the thorough mixing that occurs within the eddy.

5.4 DILUTION MODEL PREDICTIONS

RIVPLUM5 model results for the 1999 effluent flows are provided in Table 6. The proposed outfall extension would increase the acute dilution from 4.5 to 38.2 at the acute mixing zone boundary. However, even with the extended outfall the maximum allowable acute dilution would be limited to 13.8 due to the 2.5 percent of ambient flow limitation (see Section 2.3) as shown below:

$$Q_{eff} = Peak\ Day\ Effluent\ Flow = 6.38\ mgd = 9.83\ cfs$$

$$R = 2.5\%\ of\ 7Q10 = (0.025)(5030) = 125.8\ cfs$$

$$DF = Dilution\ Factor = \frac{Q_{eff} + R}{Q_{eff}}$$

$$= \frac{9.83 + 125.8}{9.83}$$

$$= 13.8$$

Therefore, the proposed open-ended outfall extension would increase the acute dilution factor from 4.5 to 13.8. The chronic dilution factor would increase from 26 to 220. Since the actual dilution factors at the downstream mixing zone boundary exceed the maximum allowable dilution as illustrated in the equation above, the lack of a need for a diffuser is confirmed.

RIVPLUM5 model results for the 2015 projected flows are provided in Table 7. The proposed outfall extension would increase chronic dilution from 11.2 with the current outfall to 95 for the proposed extension. Acute dilution would increase from 2.2 to 19.0 at the acute mixing zone boundary. However, the maximum allowable dilution would be limited to 7.3 due to the 2.5 percent of ambient flow limitation, as illustrated below:

$$Q_{eff} = 12.86\ mgd = 19.8\ cfs$$

$$R = (0.025)(5030) = 125.8\ cfs$$

$$DF = \frac{Q_{eff} + R}{Q_{eff}} = \frac{19.8 + 125.8}{19.8} = 7.3$$

Table 5. RIVPLUM5 Calibration for October 5, 1999 Tracer Study

Spread of a plume from a point source in a river with boundary effects from the shoreline
based on the method of Fischer *et al.* (1979) with correction for the effective origin of effluent

Revised 25-Nov-97

	05-Oct-99 Chronic	05-Oct-99 Acute
INPUT		
Skagit River Flow (cfs)	7810	7810
1 Effluent Discharge Rate (cfs)	4 93	4 93
2 Receiving Water Characteristics Downstream From Waste Input		
Stream Depth (ft)	8 00	8 00
Stream Velocity (fps)	0 10	0 10
Channel Width (ft)	200 0	200 0
Stream Slope (ft/ft) or Manning roughness "n"	0 007	0 007
0 if slope or 1 if Manning "n" in previous cell	0	0
3 Discharge Distance From Nearest Shoreline (ft)	0	0
4 Location of Point of Interest to Estimate Dilution		
Distance Downstream to Point of Interest (ft)	308	30 8
Distance From Nearest Shoreline (ft)	0	0
5 Transverse Mixing Coefficient Constant (usually 0 6)	0 6	0 6
6 Original Fischer Method (enter 0) or *Effective Origin* Modification (enter 1)	0	0
1 Source Conservative Mass Input Rate		
Concentration of Conservative Substance (%)	100 00	100 00
Source Conservative Mass Input Rate (cfs*%)	493 00	493 00
2 Shear Velocity		
Shear Velocity based on slope (ft/sec)	1 343	1 343
Shear Velocity based on Manning "n"		
using Prasuhn equations 8-26 and 8-54 assuming		
hydraulic radius equals depth for wide channel		
Darcy-Weisbach friction factor "f"	#N/A	#N/A
Shear Velocity from Darcy-Weisbach "f" (ft/sec)	#N/A	#N/A
Selected Shear Velocity for next step (ft/sec)	1 343	1 343
3 Transverse Mixing Coefficient (ft2/sec)	6 446	6 446
4 Plume Characteristics Accounting for Shoreline Effect (Fischer *et al.* 1979)		
Co	3 08E+00	3 08E+00
x'	4 96E-01	4 96E-02
y'o	0 00E+00	0 00E+00
y' at point of interest	0 00E+00	0 00E+00
Solution using superposition equation (Fischer eqn 5 9)		
Term for n= -2	6 32E-04	1 99E-35
Term for n= -1	2 67E-01	3 55E-09
Term for n= 0	2 00E+00	2 00E+00
Term for n= 1	2 67E-01	3 55E-09
Term for n= 2	6 32E-04	1 99E-35
Upstream Distance from Outfall to *Effective Origin* of Effluent Source (ft)	#N/A	#N/A
Effective Distance Downstream from Effluent to Point of Interest (ft)	308 00	30 80
x' Adjusted for *Effective Origin*	4 96E-01	4 96E-02
C/Co (dimensionless)	1 01E+00	2 53E+00
Concentration at Point of Interest (Fischer Eqn 5 9)	3 13E+00	7 80E+00
Unbounded Plume Width at Point of Interest (ft)	797 043	252 047
Unbounded Plume half-width (ft)	398 522	126 024
Distance from near shore to discharge point (ft)	0 00	0 00
Distance from far shore to discharge point (ft)	200 00	200 00
Plume width bounded by shoreline (ft)	200 00	126 02
Approximate Downstream Distance to Complete Mix (ft)	248	248
Theoretical Dilution Factor at Complete Mix	32 5	32 5
Calculated Flux-Average Dilution Factor Across Entire Plume Width	32 5	20 5
Calculated Dilution Factor at Point of Interest	**32 0**	**12.8**
Maximum Allowable Dilution Factor (based on volume)	397 0	40 6

Table 6. RIVPLUM5 Results for 1999 Effluent Flows

Spread of a plume from a point source in a river with boundary effects from the shoreline
based on the method of Fischer *et al.* (1979) with correction for the effective origin of effluent

Revised 25-Nov-97

	1999 Flow Shore Outfall Chronic	1999 Flow Shore Outfall Acute	1999 Flow Extended Outfall Chronic	1999 Flow Extended Outfall Acute
INPUT				
Skagit River Flow (cfs)	5030	5030	5030	5030
1 Effluent Discharge Rate (cfs)	5 41	9 83	5 41	9 83
2 Receiving Water Characteristics Downstream From Waste Input				
Stream Depth (ft)	6 75	6 75	13 00	13 00
Stream Velocity (fps)	0 09	0 09	1 80	1 80
Channel Width (ft)	400 0	400 0	400 0	400 0
Stream Slope (ft/ft) or Manning roughness "n"	0 007	0 007	0 035	0 035
0 if slope or 1 if Manning "n" in previous cell	0	0	1	1
3 Discharge Distance From Nearest Shoreline (ft)	0	0	40	40
4 Location of Point of Interest to Estimate Dilution				
Distance Downstream to Point of Interest (ft)	308	30 8	302 1	30 2
Distance From Nearest Shoreline (ft)	0	0	40	40
5 Transverse Mixing Coefficient Constant (usually 0.6)	0 6	0 6	0 6	0 6
6 Original Fischer Method (enter 0) or *Effective Origin* Modification (enter 1)	0	0	0	0
[illegible]				
1 Source Conservative Mass Input Rate				
Concentration of Conservative Substance (%)	100 00	100.00	100.00	100 00
Source Conservative Mass Input Rate (cfs*%)	540 54	982.52	540 54	982.52
2 Shear Velocity				
Shear Velocity based on slope (ft/sec)	1 233	1 233	#N/A	#N/A
Shear Velocity based on Manning "n"				
using Prasuhn equations 8-26 and 8-54 assuming				
hydraulic radius equals depth for wide channel				
Darcy-Weisbach friction factor "f"	#N/A	#N/A	0 060	0 060
Shear Velocity from Darcy-Weisbach "f" (ft/sec)	#N/A	#N/A	0 156	0 156
Selected Shear Velocity for next step (ft/sec)	1 233	1 233	0 156	0 156
3 Transverse Mixing Coefficient (ft2/sec)	4 996	4 996	1 220	1 220
4 Plume Characteristics Accounting for Shoreline Effect (Fischer *et al.*, 1979)				
Co	2 22E+00	4 04E+00	5 78E-02	1 05E-01
x'	1 07E-01	1 07E-02	1 28E-03	1 28E-04
y'o	0 00E+00	0 00E+00	1 00E-01	1 00E-01
y' at point of interest	0 00E+00	0 00E+00	1 00E-01	1 00E-01
Solution using superposition equation (Fischer eqn 5 9)				
Term for n= -2	1 10E-16	5 24E-163	0 00E+00	0 00E+00
Term for n= -1	1 72E-04	4 52E-41	0 00E+00	0 00E+00
Term for n= 0	2 00E+00	2 00E+00	1 00E+00	1 00E+00
Term for n= 1	1 72E-04	4 52E-41	1 64E-275	0 00E+00
Term for n= 2	1 10E-16	5 24E-163	0 00E+00	0 00E+00
Upstream Distance from Outfall to *Effective Origin* of Effluent Source (ft)	#N/A	#N/A	#N/A	#N/A
Effective Distance Downstream from Effluent to Point of Interest (ft)	308 00	30 80	302 10	30 20
x' Adjusted for *Effective Origin*	1 07E-01	1 07E-02	1 28E-03	1 28E-04
C/Co (dimensionless)	1 73E+00	5 46E+00	7 89E+00	2 49E+01
Concentration at Point of Interest (Fischer Eqn 5 9)	3 84E+00	2 21E+01	4 55E-01	2 62E+00
Unbounded Plume Width at Point of Interest (ft)	739 641	233 895	80 961	25 598
Unbounded Plume half-width (ft)	369 821	116 948	40 480	12 799
Distance from near shore to discharge point (ft)	0 00	0 00	40 00	40 00
Distance from far shore to discharge point (ft)	400 00	400 00	360 00	360 00
Plume width bounded by shoreline (ft)	369 82	116 95	80 48	25 60
Approximate Downstream Distance to Complete Mix (ft)	1 153	1 153	76 457	76 457
Theoretical Dilution Factor at Complete Mix	45 0	24 7	1731 6	952 7
Calculated Flux-Average Dilution Factor Across Entire Plume Width	41 5	7 2	348 4	61 0
Calculated Dilution Factor at Point of Interest	26 0	4.5	219.5	38 2
Maximum Allowable Dilution Factor (based on volume)	233 6	13 8	233 6	13.8

Table 7. RIVPLUM5 Results for 2015 Effluent Flows

Spread of a plume from a point source in a river with boundary effects from the shoreline
b... d on the method of Fischer *et al.* (1979) with correction for the effective origin of effluent

Revised 25-Nov-97

	2015 Flow Shore Outfall Chronic	2015 Flow Shore Outfall Acute	2015 Flow Extended Outfall Chronic	2015 Flow Extended Outfall Acute
INPUT				
Skagit River Flow (cfs)	5030	5030	5030	5030
1 Effluent Discharge Rate (cfs)	12 52	19 80	12 52	19 80
2 Receiving Water Characteristics Downstream From Waste Input				
Stream Depth (ft)	6 75	6 75	13 00	13 00
Stream Velocity (fps)	0 09	0 09	1 80	1 80
Channel Width (ft)	400 0	400 0	400 0	400 0
Stream Slope (ft/ft) or Manning roughness "n"	0 007	0 007	0 035	0 035
0 if slope or 1 if Manning "n" in previous cell	0	0	1	1
3 Discharge Distance From Nearest Shoreline (ft)	0	0	40	40
4 Location of Point of Interest to Estimate Dilution				
Distance Downstream to Point of Interest (ft)	308	30 8	302 1	30 2
Distance From Nearest Shoreline (ft)	0	0	40	40
5 Transverse Mixing Coefficient Constant (usually 0 6)	0 6	0 6	0 6	0 6
6 Original Fischer Method (enter 0) or *Effective Origin* Modification (enter 1)	0	0	0	0
[illegible]				
1 Source Conservative Mass Input Rate				
Concentration of Conservative Substance (%)	100 00	100 00	100 00	100 00
Source Conservative Mass Input Rate (cfs*%)	1,252 02	1,980 44	1,252 02	1,980 44
2 Shear Velocity				
Shear Velocity based on slope (ft/sec)	1 233	1 233	#N/A	#N/A
Shear Velocity based on Manning "n" using Prasuhn equations 8-26 and 8-54 assuming hydraulic radius equals depth for wide channel				
Darcy-Weisbach friction factor "f"	#N/A	#N/A	0 060	0 060
Shear Velocity from Darcy-Weisbach "f" (ft/sec)	#N/A	#N/A	0 156	0 156
Selected Shear Velocity for next step (ft/sec)	1 233	1 233	0 156	0 156
3 Transverse Mixing Coefficient (ft2/sec)	4 996	4 996	1 220	1 220
4 Plume Characteristics Accounting for Shoreline Effect (Fischer *et al.* 1979)				
Co	5 15E+00	8 15E+00	1 34E-01	2 12E-01
x'	1 07E-01	1 07E-02	1 28E-03	1 28E-04
y'o	0 00E+00	0 00E+00	1 00E-01	1 00E-01
y' at point of interest	0 00E+00	0 00E+00	1 00E-01	1 00E-01
Solution using superposition equation (Fischer eqn 5 9)				
Term for n= -2	1 10E-16	5 24E-163	0 00E+00	0 00E+00
Term for n= -1	1 72E-04	4 52E-41	0 00E+00	0 00E+00
Term for n= 0	2 00E+00	2 00E+00	1 00E+00	1 00E+00
Term for n= 1	1 72E-04	4 52E-41	1 64E-275	0 00E+00
Term for n= 2	1 10E-16	5 24E-163	0 00E+00	0 00E+00
'pstream Distance from Outfall to *Effective Origin* of Effluent Source (ft)	#N/A	#N/A	#N/A	#N/A
Effective Distance Downstream from Effluent to Point of Interest (ft)	308 00	30 80	302 10	30 20
x' Adjusted for *Effective Origin*	1 07E-01	1 07E-02	1 28E-03	1 28E-04
C/Co (dimensionless)	1 73E+00	5 46E+00	7 89E+00	2 49E+01
Concentration at Point of Interest (Fischer Eqn 5 9)	8 89E+00	4 45E+01	1 06E+00	5 28E+00
Unbounded Plume Width at Point of Interest (ft)	739 641	233 895	80 961	25 598
Unbounded Plume half-width (ft)	369 821	116 948	40 480	12 799
Distance from near shore to discharge point (ft)	0 00	0 00	40 00	40 00
Distance from far shore to discharge point (ft)	400 00	400 00	360 00	360 00
Plume width bounded by shoreline (ft)	369 82	116 95	80 48	25 60
Approximate Downstream Distance to Complete Mix (ft)	1 153	1 153	76 457	76 457
Theoretical Dilution Factor at Complete Mix	19 4	12 3	747 6	472 6
Calculated Flux-Average Dilution Factor Across Entire Plume Width	[illegible]	3 6	150 4	30 2
Calculated Dilution Factor at Point of Interest	11 2	2.2	94.8	19 0
Max um Allowable Dilution Factor (based on volume)	101 4	7 3	101 4	7.3

SECTION 6:
WATER QUALITY-BASED EFFLUENT LIMITATIONS

6.1 REASONABLE POTENTIAL TO EXCEED WATER QUALITY STANDARDS

The *reasonable potential to exceed water quality standards* is a standard statistical test developed by EPA and adopted by Ecology (1992) to establish the need for effluent limitations in NPDES permits. The method establishes a maximum expected concentration for toxicants based on effluent data available for the discharge. The predicted concentration at acute and chronic mixing zone boundaries is determined based on the dilution factors and ambient concentrations. If acute and chronic water quality standards are met by this test, then no effluent limitations are required to be placed in the NPDES permit. If there is a reasonable potential to exceed standards, then a permit limit for that parameter is required.

Table 8 displays the reasonable potential evaluation for the 1999 flows and existing effluent data for toxicants. The results demonstrate that effluent limits will be required for ammonia, copper, lead, silver and zinc in the next NPDES permit if the outfall is not extended. If the outfall is extended, only silver would have a reasonable potential to exceed water quality standards.

Table 9 shows the reasonable potential evaluation for the 2015 flows and existing effluent data. The results indicate that effluent limits will be required for all the detected toxicants in the next NPDES permit if the outfall remains unchanged. If the outfall is extended, only copper, lead and silver would have a reasonable potential to exceed water quality standards.

Table 8 **Reasonable Potential to Exceed Water Quality Standards – 1999 Effluent Flows**

Dry Season Flow - Existing Outfall											
							Mixing Zone		WQ Std.		
Toxicant	Max Eff	CV	n	Mult.	Ambient	MEC	Acute	Chronic	Acute	Chronic	Limit Req'd?
Cl2	50	0.6	730	1.0	0	50	11.1	1.9	13	7.5	no
NH3-N	45400	0.6	104	1.0	22	45400	10106	1767	8314	1877	YES
Cu	37	0.45	68	1.0	0.55	37	8.65	1.95	4.61	3.47	YES
Hg	ND								2.1	0.012	no
Pb	19	0.6	12	2.8	0.02	53.2	11.84	2.07	13.9	0.54	YES
Ag	3	0.6	12	2.8	0	8.4	1.87		0.32		YES
Zn	64	0.6	12	2.8	9.3	179.2	47.1	15.8	35.4	32.3	YES

Acute Dilution = 4.5
Chronic Dilution = 26

Dry Season Flow - Proposed Outfall Extension											
							Mixing Zone		WQ Std.		
Toxicant	Max Eff	CV	n	Mult.	Ambient	MEC	Acute	Chronic	Acute	Chronic	Limit Req'd?
Cl2	50	0.6	730	1.0	0	50	3.6	0.2	13	7.5	no
NH3-N	45400	0.6	104	1.0	22	45400	3310	229	8314	1877	no
Cu	37	0.45	68	1.0	0.55	37	3.19	0.72	4.61	3.47	no
Hg	ND								2.1	0.012	no
Pb	19	0.6	12	2.8	0.02	53.2	3.87	0.26	13.9	0.54	no
Ag	3	0.6	12	2.8	0	8.4	0.61	0.04	0.32		YES
Zn	64	0.6	12	2.8	9.3	179.2	21.6	10.1	35.4	32.3	no

Acute Dilution = 13.8
Chronic Dilution = 219

All units are µg/L

Table 9 **Reasonable Potential to Exceed Water Quality Standards – 2015 Effluent Flows**

Dry Season Flow - Existing Outfall											
							Mixing Zone		WQ Std.		
Toxicant	Max Eff	CV	n	Mult.	Ambient	MEC	Acute	Chronic	Acute	Chronic	Limit Req'd?
Cl2	50	0.6	730	1.0	0	50	22.7	4.5	13	7.5	YES
NH3-N	45400	0.6	104	1.0	22	45400	20648	4074	8314	1877	YES
Cu	37	0.45	68	1.0	0.55	37	17.12	3.80	4.61	3.47	YES
Hg	ND								2.1	0.012	no
Pb	19	0.6	12	2.8	0.02	53.2	24.19	4.77	13.9	0.54	YES
Ag	3	0.6	12	2.8	0	8.4	3.82		0.32		YES
Zn	64	0.6	12	2.8	9.3	179.2	86.5	24.5	35.4	32.3	YES

Acute Dilution = 2.2
Chronic Dilution = 11.2

Dry Season Flow - Proposed Outfall Extension											
							Mixing Zone		WQ Std.		
Toxicant	Max Eff	CV	n	Mult.	Ambient	MEC	Acute	Chronic	Acute	Chronic	Limit Req'd?
Cl2	50	0.6	730	1.0	0	50	6.8	0.5	13	7.5	no
NH3-N	45400	0.6	104	1.0	22	45400	6238	501	8314	1877	no
Cu	37	0.45	68	1.0	0.55	37	5.54	0.93	4.61	3.47	YES
Hg	ND								2.1	0.012	no
Pb	19	0.6	12	2.8	0.02	53.2	7.30	0.58	13.9	0.54	YES
Ag	3	0.6	12	2.8	0	8.4	1.15	0.09	0.32		YES
Zn	64	0.6	12	2.8	9.3	179.2	32.6	11.1	35.4	32.3	no

Acute Dilution = 7.3
Chronic Dilution = 94.8

All units are µg/L

6.2 NPDES PERMIT LIMITS

NPDES permit effluent limits are derived using methods developed by EPA and adopted by Ecology (1992). Permit limit worksheets are provided in Appendix E. The effluent limits for toxicants with a reasonable potential to exceed standards are summarized in Table 10. These limits would apply during the months July through October when river discharge is low, and would be in addition to ammonia and BOD limitations derived from the TMDL. Limits for 1999 and 2015 effluent flows are shown separately.

Table 10 Water Quality-Based NPDES Permit Limits (July-October)

1999 FLOWS				
	Existing Outfall		**Proposed Outfall Extension**	
	Daily Maximum	**Monthly Average**	**Daily Maximum**	**Monthly Average**
Ammonia-N, mg/L	37.3	16.6		
Copper, µg/L	18.8	12.9		
Lead, µg/L	22.2	15.2		
Silver, µg/L	1.4	1.0	4.4	3.0
Zinc, µg/L	127	87		

2015 FLOWS				
	Existing Outfall		**Proposed Outfall Extension**	
	Daily Maximum	**Monthly Average**	**Daily Maximum**	**Monthly Average**
Ammonia-N, mg/L	18.3	8.1		
Copper, µg/L	9.5	6.5	30.2	20.7
Lead, µg/L	9.6	6.6	81	55
Silver, µg/L	0.7	0.5	2.3	1.6
Zinc, µg/L	66	46		

NOTE: Blanks indicate no limit required.

SECTION 7:
REFERENCES

Ecology, 1992. *Permit Writer's Manual.* Washington State Department of Ecology Publication No. 92-109.

Ecology, 1998. *Criteria for Sewage Works Design (Orange Book).* Washington State Department of Ecology.

Pickett, 1997. *Lower Skagit River Total Maximum Daily Load Water Quality Study.* Washington State Department of Ecology Publication No. 97-326a.

Appendix A

Ambient Water Quality Data and
Ammonia Criteria Worksheet

COSMOPOLITAN
ENGINEERING
GROUP

Civil, Environmental,

and Recreational

Consulting

WASHINGTON STATE
DEPT. OF ECOLOGY

/ Conditions & Trends / River and Stream WQ Monitoring

Station 03A060

AMBIENT MONITORING DATA

Station 03A060
Water Quality Data Summary - Metals

Last updated 28-May-1998

1995 (03A060)	Mercury (micrograms/ Liter)	Cadmium (micrograms/ Liter)	Zinc (micrograms/ Liter)	Lead (micrograms/ Liter)	Chromium (micrograms/ Liter)	Copper (micrograms/ Liter)	Nickel (micrograms/ Liter)	Hardness** (milligrams/ Liter)
03/22/1995		0.030u	0.400u	0.020u		0.460p	0.689	28
01/18/1995		0.020u	0.550p	0.020u		0.360p	0.620	29

*Data qualifiers: u, j - estimated value k - actual value known to be less p - too numerous to count v - contamination in the blankt x - high background count

1994 (03A060)	Mercury (micrograms/ Liter)	Cadmium (micrograms/ Liter)	Zinc (micrograms/ Liter)	Lead (micrograms/ Liter)	Chromium (micrograms/ Liter)	Copper (micrograms/ Liter)	Nickel (micrograms/ Liter)	Hardness** (milligrams/ Liter)
11/16/1994		0.040u	1.900p	0.020u		0.546	1.000u	27
09/20/1994		0.040u	1.000u	0.020u		0.280p	1.000u	23
07/19/1994		0.040u	9.340p	0.020u		0.290p	1.000u	20
05/17/1994		0.040u	1.000u	0.023p		0.348p	1.000u	22

*Data qualifiers: u, j - estimated value k - actual value known to be less p - too numerous to count v - contamination in the blankt x - high background count

** Hardness (salt concentration) is factored into the water quality criteria for all of the metals except mercury.

Washington State Department of Ecology
Please send comments to stba461@ecy.wa.gov

WASHINGTON STATE
DEPT. OF ECOLOGY

/ Conditions & Trends / River and Stream WQ Monitoring

AMBIENT MONITORING DATA

Station 03A060
Six Year Water Quality Data Summary

Last updated 26-May-1998

Metals data is also available for this station

Station name	Class	Latitude	Longitude	Elevation (ft)	River mile	Watershed(s)	External mapping li
Skagit R nr Mount Vernon	A	48 26 42.0	122 20 03.0	14	15.9	Skagit/Stillaguamish	Tiger Mapping Service M

Years-monitored history *Note: Data for years not presented here are available on request: See Requesting additional data.*

98	97	96	95	94	93	92	91	90	89	88	87	86	85	84	83	82	81	80	79	78	77	76	75	74	73	72	71	70	69	68	67	66	65	64	63	62	61	60	59
•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•	•				•	•		•	•	•	•	•	•	•	•	•

1997 (03A060)	Time	Flow (CFS)	Temperature (C)	Conductivity (umhos/25c)	Oxygen (mg/L)	Oxygen Saturation (%)	pH	Fecal Coliforms (colonies/100ml)	Suspended Solids (mg/L)	Total Persulfate Nitrogen (mg/L)	Ammonia Nitrogen (mg/L)	Total Phosphorus (mg/L)	Dissolved Soluble Phosphorus (mg/L)
09/23/19	0730	11300	12.6	44	10.4	96.4	7.4	15	14	0.079	0.010u	0.028	0.005u
08/20/1997	0720	13300	13.0	42	10.3	97.3	7.6	3	23	0.042	0.013	0.028	0.005u
07/23/1997	0735	26200	10.6	39	10.4	92.0	7.5	14	31	0.067	0.010u	0.038	0.005u
06/18/1997	0735	57800	8.0	32	11.7	97.7	7.7	130j	246	0.198j	0.010uj	0.052j	0.006
05/21/1997	0740	28400	6.6	40	11.6	93.9	7.5	11	32	0.088	0.011	0.045	0.005u
04/23/1997	0725	24700	6.5	52	11.6	93.7	7.6	3	27	0.213	0.038	0.068	0.005u
03/19/1997	0735	38400	4.2	39	12.6	95.6	7.2	32	389	0.480	0.023	0.275j	0.005u
02/19/1997	0750	25600	4.0	52	12.6	95.7	7.4	40	26	0.241	0.010u	0.078	0.005
01/22/1997	0805	29600	5.1	47	12.3	96.0	7.4	4s	53	0.210	0.010u	0.044	0.005u

■ *result fails water quality criteria* Data qualifiers: u, j - estimated value k - actual value known to be less s - spreader x - high background

1996 (03A060)	Time	Flow (CFS)	Temperature (C)	Conductivity (umhos/25c)	Oxygen (mg/L)	Oxygen Saturation (%)	pH	Fecal Coliforms (colonies/100ml)	Suspended Solids (mg/L)	Total Persulfate Nitrogen (mg/L)	Ammonia Nitrogen (mg/L)	Total Phosphorus (mg/L)	Dissolved Soluble Phosphorus (mg/L)
12/17/1996	0825	13900	5.3	61	12.5	96.7	7.5	3	11	0.227	0.014	0.027	0.005u
11/20/1996	0825	16200	4.3	55	12.1	92.0	7.7	8	10	0.158	0.010u	0.010u	0.005u

10/23/1996	0805	21000	7.5	41	11.2	92.3	7.4	73	48	0.211	0.010u	0.046	0.005u
09/18/1996	0740	9960	11.6	76	10.8	97.3	7.4	14	10	0.077	0.010u	0.014	0.005u
08/21/1996	0725	7280	13.5	53	10.1	94.9	7.5	15	9	0.068	0.010u	0.022	0.005u
07/24/1996	0750	15500	14.6	44	10.2	99.5	7.6	13x	11	0.033	0.010u	0.014	0.005u
06/19/1996	0715	16800	10.3	46	11.3	99.2	7.6	5	8	0.074	0.010u	0.015	0.005u
05/22/1996	0800	17700	8.5	52	11.6	98.6	7.3	20	10	0.121	0.010u	0.010u	0.005u
04/24/1996	0705	36100	6.9	41	11.7	96.0	7.4	59	340	0.245	0.010u	0.252	0.005u
03/20/1996	0745	16000	6.3	60	12.2	97.5	7.1	10	11	0.119	0.010u	0.014	0.005u
02/21/1996	0735	26200	5.6	47	12.2	97.9	7.7	2	36	0.160	0.010u	0.037	0.005u
01/24/1996	0810	19200	4.1	58	12.4	95.2	7.3	22	7	0.208	0.010u	0.016	0.005u

■ *result fails water quality criteria* Data qualifiers: u, j - estimated value k - actual value known to be less s - spreader x - high background

1995 (03A060)	Time	Flow (CFS)	Temperature (C)	Conductivity (umhos/ 25c)	Oxygen (mg/L)	Oxygen Saturation (%)	pH	Fecal Coliforms (colonies/ 100ml)	Suspended Solids (mg/L)	Total Persulfate Nitrogen (mg/L)	Ammonia Nitrogen (mg/L)	Total Phosphorus (mg/L)	Dissolved Soluble Phosphorus (mg/L)
12/19/1995	0735	25100	6.2	52	12.0	96.0	7.5	4	38	0.159	0.010u	0.020	0.005u
11/21/1995	0745	34500	7.0	58	11.6	93.5	7.2	7s	90	0.185	0.010u	0.079	0.005u
10/18/1995	0745	32800	8.9	39	11.2	93.6	7.4	40	100	0.227	0.010u	0.076	0.005u
09/20/1995	0905	8700	13.7	55	10.2	95.2	7.6	27	14	0.075	0.010u	0.010u	0.007
08/23/1995	0910	9460	14.3	54	10.2	97.2	7.7	11	8	0.091	0.010u	0.010u	0.005u
07/19/1995	0900	13500	15.2	40	9.9	96.6	8.1	16	21	0.083	0.022	0.019	0.005u
06/21/1995	0910	13100	11.0	45	11.1	99.3	7.6	20	6	0.106	0.010u	0.018	0.005u
05/17/1995	0740	24600	9.1	42	11.2	96.2	7.5	10	383	0.080	0.010u	0.010u	0.005u
04/19/1995	0905	10100	7.1	67	11.9	98.0	7.2	8	8	0.144	0.010u	0.022	0.005u
03/22/1995	0900	21200	5.1	54	12.2	97.2	7.2	8s	11	0.163	0.010u	0.010	0.005u
02/22/1995	0930	40800	4.8	43	12.6	96.4		26s	95	0.203	0.010u	0.082	0.005u
01/18/1995	0930	18700	5.0	60	12.4	97.4	7.3	6s	12	0.214	0.010u	0.048	0.005u

■ *result fails water quality criteria* Data qualifiers: u, j - estimated value k - actual value known to be less s - spreader x - high background

1994 (03A060)	Time	Flow (CFS)	Temperature (C)	Conductivity (umhos/ 25c)	Oxygen (mg/L)	Oxygen Saturation (%)	pH	Fecal Coliforms (colonies/ 100ml)	Suspended Solids (mg/L)	Total Persulfate Nitrogen (mg/L)	Ammonia Nitrogen (mg/L)	Total Phosphorus (mg/L)	Dissolved Soluble Phosphorus (mg/L)
12/21/1994	0950	51700	5.0	36	12.4	96.0	7.4	54s	248	0.266	0.010u	0.210	0.005u
11/16/1994	1100	13700	7.5	58	11.5	96.4	7.4	13	15	0.197	0.010u	0.010u	0.010k
10/19/1994	1010	4340	10.4	67	10.8	95.7	7.5	5	3	0.094	0.010u	0.010k	0.010k
09/20/1994	1440	8560	14.9	54	10.4	101.6	7.5	59	36	0.075	0.010k	0.026	0.010k
08/16/1994	1335	10600	15.7	46	10.1	100.7	7.6	31	146	0.085	0.010k	0.103	0.010k
07/19/1994	1355	11700	15.1	41	10.3	101.4		1	13	0.079	0.010k	0.014	0.010k

06/21/1994	1350	13500	14.2	44	10.3	99.3	7.5	1	6	0.079	0.010k	0.010k	0.010k	:
05/17/1994	1445	15300	12.1	46	10.1	93.2	7.3	15	7	0.010k	0.010k	0.010k	0.010k	:
04/19/1994	1410	19700	8.1	40	11.3	94.5	7.4	21	18	0.124	0.010k	0.010k	0.010k	(
03/22/1994	1345	15600	4.2	58	12.4	95.1	7.4	22	13	0.271	0.010k	0.010k	0.010k	:
02/22/1994	1405	10400	5.5	65	12.6	99.4	7.5	15	7	0.325	0.012	0.011	0.010k	:
01/1. 994	1330	17500	5.7	59	12.1	95.0		2	12	0.240	0.010k	0.010k	0.010k	‹

■ *result fails water quality criteria* Data qualifiers: u, j - estimated value k - actual value known to be less s - spreader x - high background

1993 (03A060)	Time	Flow (CFS)	Temperature (C)	Conductivity (umhos/ 25c)	Oxygen (mg/L)	Oxygen Saturation (%)	pH	Fecal Coliforms (colonies/ 100ml)	Suspended Solids (mg/L)	Total Persulfate Nitrogen (mg/L)	Ammonia Nitrogen (mg/L)	Total Phosphorus (mg/L)	Dissolved Soluble Phosphorus (mg/L)	(
12/20/1993	1335	12600	5.6	61	12.4	97.0	7.8	1	9	0.268	0.045	0.010k	0.010k	
11/16/1993	1510	8510	8.1	60	11.4	95.6	7.5	15	7	0.143	0.010k	0.010k	0.010k	
10/19/1993	1340	5570	11.4	60	11.0	98.5	7.6	4	4	0.150	0.011	0.010k	0.010k	:
09/22/1993	0850	7430	11.7	52	11.1	100.1	7.8	10	10				0.010k	:
08/18/1993	0855	9390	14.2	45	10.4	99.6	7.4	34	32		0.011	0.023	0.010k	
07/21/1993	0840	21500	12.0	30	10.6	97.1	7.3	[redacted]	111		0.010k	0.036	0.010k	
06/23/1993	0930	16500	10.2	37	11.0	95.8	7.3	12	14		0.010k	0.010k	0.010k	:
05/19/1993	0830	26800	10.7	32	11.3	100.8	8.1	15	74		0.010k	0.031	0.010k	
04/21/1993	0900	8720	8.2	58	11.4	95.8	7.8	4	7		0.013	0.010k	0.010	
03/17/1993	0845	10100	4.6	55	12.4	96.5	7.5	5	11		0.020	0.018	0.010k	:
02/17/1993	0925	12200	1.4	59	13.5	94.0	7.3	3k	12		0.015	0.015	0.010k	
01/20/1993	0850	9190	3.5	67	13.2	101.7	7.6	3	23		0.015	0.015	0.010k	

■ *result fails water quality criteria* Data qualifiers: u, j - estimated value k - actual value known to be less s - spreader x - high background

1992 (03A060)	Time	Flow (CFS)	Temperature (C)	Conductivity (umhos/ 25c)	Oxygen (mg/L)	Oxygen Saturation (%)	pH	Fecal Coliforms (colonies/ 100ml)	Suspended Solids (mg/L)	Total Persulfate Nitrogen (mg/L)	Ammonia Nitrogen (mg/L)	Total Phosphorus (mg/L)	Dissolved Soluble Phosphorus (mg/L)	(
12/16/1992	0910	11900	5.0	57	12.8	99.3	7.4	4	25		0.010	0.012	0.010k	:
11/18/1992	0840	14200	9.0	48	11.3	96.2	7.5	6	33		0.010k	0.020	0.010k	‹
10/21/1992	0920	11700	10.9	45	10.5	94.9	7.3	84	84		0.027	0.049	0.010k	
09/23/1992	0745	6600	14.1	44	9.9	96.1	7.6	43x	77		0.044	0.061	0.010k	:
08/19/1992	0850	9680	15.4	46	9.8	96.1	7.4	11	1230		0.010k	0.737	0.010	:
07/22/1992	0750	11700	13.4	43	9.9	94.3	7.2	80	130		0.026	0.095	0.010k	:
06/17/1992	1050	10700	11.0	50	10.7	95.4	7.3	13	7		0.012	0.015	0.010k	
05/20/1992	1020	15400	10.1	43	11.1	97.2	7.1	8	6		0.010k	0.010k	0.010k	:
04/22/1992	0950	10300	8.5	52	11.4	96.0	7.4	51	17		0.014	0.019	0.010k	:
03/18/1992	0950	13500	6.4	58	11.8	93.8	7.4	4	8		0.010k	0.010	0.010k	:

02/19/1992	0910	17800	5.0	55	12.1	93.8	7.2	6	12		0.019	0.013	0.010k
01/22/1992	0845	16300	5.2	57	12.5	96.7	7.4	5	2		0.016	0.019	0.010k

■ *result fails water quality criteria* Data qualifiers: u, j - estimated value k - actual value known to be less s - spreader x - high background

1991 (03A060)	Time	Flow (CFS)	Tem-perature (C)	Conduc-tivity (umhos/ 25c)	Oxygen (mg/L)	Oxygen Satura-tion (%)	pH	Fecal Coliforms (colonies/ 100ml)	Sus-pended Solids (mg/L)	Total Persulfate Nitrogen (mg/L)	Ammonia Nitrogen (mg/L)	Total Phosphorus (mg/L)	Dissolved Soluble Phosphorus (mg/L)
12/11/1991	0740	22900	6.2	55	11.8	95.8	7.1	13	21		0.020	0.024	0.010k
11/13/1991	0810	26300	9.8	39	11.4	99.8	7.3	37	120		0.041	0.083	0.010k
10/23/1991	0800	7300	9.7	50	11.1	96.3	7.3	32	13		0.011	0.014	0.010k

■ *result fails water quality criteria* Data qualifiers: u, j - estimated value k - actual value known to be less s - spreader x - high background

Washington State Department of Ecology
Please send comments to stba461@ecy.wa.gov

AMBIENT DATA AND AMMONIA-N CRITERIA (SUMMER)

Date	Ambient Temp (C)	pH	NH3-N (ug/L)	Fta	FTc	FPH	RATIO	pKa	fraction	Acute NH3-N Criteria (ug/L)	Chronic NH3-N Criteria (ug/L)
23-Sep-97	12.6	7.4	10	1.667	1.667	1.600	20.202	9.642	0.569%	14,068	2,143
20-Aug-97	13	7.6	13	1.622	1.622	1.305	15.631	9.629	0.927%	10,890	2,144
23-Jul-97	10.6	7.5	10	1.914	1.914	1.435	17.886	9.709	0.614%	12,673	2,180
23-Oct-96	7.5	7.4	10	2.371	2.371	1.600	20.202	9.816	0.383%	14,722	2,242
18-Sep-96	11.6	7.4	10	1.786	1.786	1.600	20.202	9.676	0.527%	14,177	2,159
21-Aug-96	13.5	7.5	10	1.567	1.567	1.435	17.886	9.612	0.767%	12,395	2,132
24-Jul-96	14.6	7.6	10	1.452	1.452	1.305	15.631	9.576	1.047%	10,778	2,122
18-Oct-95	8.9	7.4	10	2.153	2.153	1.600	20.202	9.767	0.427%	14,518	2,211
20-Sep-95	13.7	7.6	10	1.545	1.545	1.305	15.631	9.605	0.978%	10,839	2,134
23-Aug-95	14.3	7.7	10	1.483	1.483	1.201	13.500	9.586	1.285%	9,343	2,129
19-Jul-95	15.2	8.1	22	1.393	1.413	1.000	13.500	9.556	3.382%	4,536	1,020
19-Oct-94	10.4	7.5	10	1.941	1.941	1.435	17.886	9.716	0.604%	12,695	2,184
20-Sep-94	14.9	7.5	10	1.422	1.422	1.435	17.886	9.566	0.852%	12,284	2,113
16-Aug-94	15.7	7.6	10	1.346	1.413	1.305	15.631	9.540	1.136%	10,711	2,009
19-Jul-94	15.1	7.6	10	1.403	1.413	1.305	15.631	9.559	1.087%	10,746	2,101
19-Oct-93	11.4	7.6	11	1.811	1.811	1.305	15.631	9.682	0.821%	11,020	2,169
22-Sep-93	11.7	7.8	10	1.774	1.774	1.118	13.500	9.672	1.324%	8,132	1,854
18-Aug-93	14.2	7.4	11	1.493	1.493	1.600	20.202	9.589	0.643%	13,912	2,119
21-Jul-93	12	7.3	10	1.738	1.738	1.807	22.518	9.662	0.433%	15,735	2,150
21-Oct-92	10.9	7.3	27	1.875	1.875	1.807	22.518	9.699	0.397%	15,877	2,170
23-Sep-92	14.1	7.6	44	1.503	1.503	1.305	15.631	9.592	1.008%	10,811	2,128
19-Aug-92	15.4	7.4	10	1.374	1.413	1.600	20.202	9.549	0.704%	13,809	2,046
22-Jul-92	13.4	7.2	26	1.578	1.578	2.068	24.773	9.615	0.383%	17,114	2,126
23-Oct-91	9.7	7.3	11	2.037	2.037	1.807	22.518	9.740	0.362%	16,046	2,193

	Temp (C)	pH	NH3-N (ug/L)
100%-ile	15.7	8.1	44.0
95%-ile	15.4	7.8	26.9
90%-ile	15.2	7.7	24.8

	Acute NH3-N Criteria (ug/L)	Chronic NH3-N Criteria (ug/L)
0%-ile	4,536	1,020
5%-ile	8,314	1,877
10%-ile	9,753	2,020

COSMOPOLITAN ENGINEERING GROUP

Appendix B

Inspection Dive Report

Civil, Environmental, and Recreational Consulting

Appendix C

Tracer Study Data

COSMOPOLITAN ENGINEERING GROUP

Civil, Environmental, and Recreational Consulting

Printed on Recycled Paper

ENGINEERING GROUP

117 South 8th Street
Tacoma, WA 98402

Phone (253) 272-7220
Fax (253) 272-7250

Memorandum

DATE: September 21, 1999

TO: File MTV001

FROM: Bill Fox, Cosmopolitan Engineering

RE: Mount Vernon WWTP Outfall Inspection Dive

Merita Trohimovich and I conducted a dive inspection of the Mount Vernon WWTP outfall at 16:30 on September 20, 1999. The river discharge was 8870 cfs and stage was 11.58 ft measured at USGS station 12200500 near Mount Vernon.

The outfall is located within a small depression in the riverbank. There is a clearly visible eddy formed in the depression. The effluent was more turbid than the ambient water, and was visible within the eddy.

The configuration of the outfall, adjacent shoreline, eddy and visible effluent plume are shown on the attached drawing.

<u>Dive Log</u>

Current speed during the dive was 2 to 3 fps. We entered the water from upstream and conducted the inspection from a tether line.

Visibility was 6 ft outside the effluent plume, and near zero inside the plume.

The outfall was easily found, in excellent condition, and armored with riprap generally as shown on the drawings. The outfall is 24-inch class 5 ductile iron pipe with a plain end. There was only about 3 feet showing at the crown and none at the invert.

The pipe invert was 8 ft deep. There was a flat shelf for approximately 15 feet offshore, with a sharp drop-off to a depth of 18 feet. There has been some cutting of the deeper section of riverbank that has reduced the horizontal apron offshore of the outfall.

The riverbank was a relatively stable combination of riprap and mud. The riverbed offshore of the drop-off was a uniform coarse sand and rounded gravel bottom, which was clearly mobile sediment.

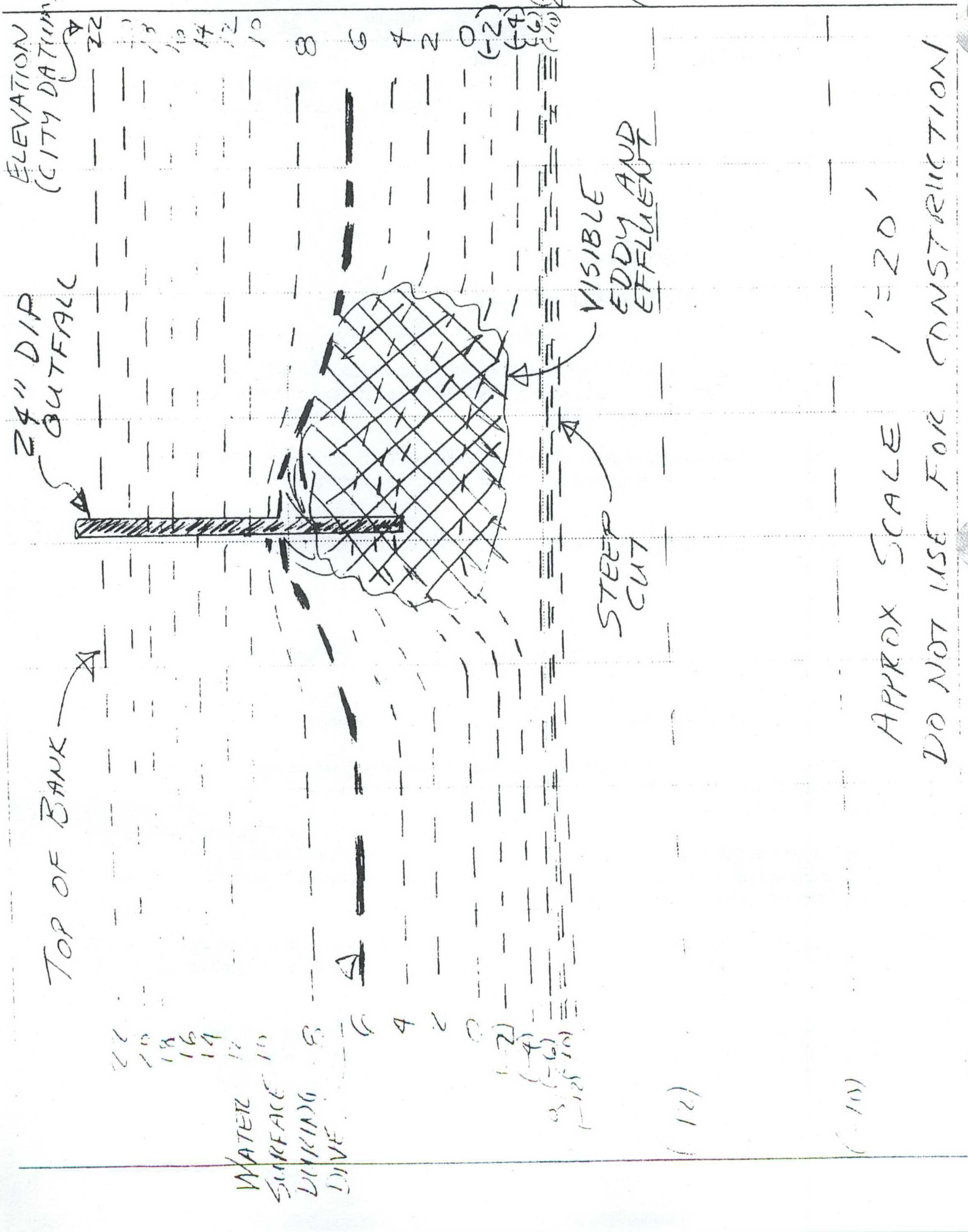
Global diving inspected the outfall in 1995 and found an 8-inch diameter log about 1 ft offshore of the pipe terminus. The same obstruction was observed in the same configuration during our inspection. There was considerable wood debris upstream and (especially) downstream of the outfall.

COSMOPOLITAN ENGINEERING GROUP

Job Name Mt. Vernon Dive Report Sheet (10) of

Job Number MTV001 Checked

By FOX Date 9/21/99 Date



Effluent Dye
Injection 10/5/98

Approx 0.5 L Rhodamine
WT @ 23.8% Sol'n
disch to CCC wier
overflow over 5.5 min

Effl. Q = 3.2 mgd
Time = 1233 start
Effl sample # 24, 40

River/mz Samples 12:38 12:45 ±

Sample Sta	Bottle #	Desc
1	21	b.g.
2	22	u.s. amz
3	23	outfall
4	25	d.s. amz
5	26	Between piling + shore
6	27	cmz (300' ±)
4	28	
2	29	

Photos 12:45-48 ±

Large Cone - outfall
Small Cones - us + ds amz

Dye o/b, dispersing 12:58

" " 13:04

River Flows

1200 hrs 10/4/99 Q = 6590 cfs

1245 hrs 10/5/99 Q = 7810 cfs

10/6/98

MTV/OU1

Dye Study Results from 10/5

Station	Bottle #	Conc	Dilution
Effl.	24	114	—
Effl.	40	120	—
1	21	0	
2	22	10.1	11.6
3	23	24.4	4.8
4	25	9.6	12.2
5	26	9.9	11.8
6	27	1.8	65
4	28	8.6	13.6
2	29	9.7	12.1

Note: Fluoro zeroed to sample #21

WWTP Effluent

```
* Sea-Bird SBE 19 Data File:
* FileName = \SBE19\mtv00.HEX
* Software Version 4.233
* Temperature SN = 2678
* Conductivity SN = 2678
* System UpLoad Time = Oct 05 1999 17:15:56
* ds
* SEACAT PROFILER V3.1b SN 2678   10/05/99  17:16:13.118
* strain gauge pressure sensor: S/N = 184559,  range = 100 psia,
tc = -183
* Narrow Range Conductivity
* clk = 32768.547   iop = 176   vmain = 11.5   vlith = 5.0
* mode = PROFILE  ncasts = 6
* sample rate = 1 scan every 2.0 seconds
* minimum raw conductivity frequency for pump turn on = 2727 hertz
* pump delay = 33 seconds
* samples = 440   free = 115645   lwait = 10 msec
* SW1 = C8   battery cutoff = 7.3 volts
* number of voltages sampled = 2
* logdata = NO
* S>
* cast 0  10/05  11:34:39   samples 0 to 65 sample rate = 1 scan e
very 2.0 seconds  stop = switch off
# nquan = 6
# nvalues = 66
# units = metric
# name 0 = scan: scan number
# name 1 = t068: temperature, IPTS-68 [deg C]
# name 2 = sal: salinity, PSS-78 [PSU]
# name 3 = pr: pressure [db]
# name 4 = oxMg/L: oxygen [mg/l]
# name 5 = flag:  0.000e+00
# span 0 = 1, 66
# span 1 = 15.2434, 19.6248
# span 2 = 0.0169, 0.4008
# span 3 = -0.331, 0.642
# span 4 = -16.77677, 37.60287
# span 5 = 0.000e+00, 0.000e+00
# interval = seconds: 0.5
# start_time = Oct 05 1999 11:34:39
# bad_flag = -9.990e-29
# sensor 0 = Frequency 0  temperature, 2678, 12 Jun 99
# sensor 1 = Frequency 1  conductivity, 2678, 12 Jun 99, cpcor = -
9.5700e-08
# sensor 2 = Extrnl Volt  0  oxygen, current, 230792, 16 Jun 99
# sensor 3 = Extrnl Volt  1  oxygen, temperature, 230792, 16 Jun 9
9
# sensor 4 = Pressure Voltage, 184559, 15 Jun 99
```

datcnv_date = Oct 05 1999 17:20:21, 4.233
datcnv_in = MTV00.HEX SBE19.CON
datcnv_skipover = 0
file_type = ascii
END

	Temp (c)				
1	18.7773	0.0920	-0.304	-0.14406	0.000e+00
2	19.0572	0.1915	-0.016	-0.14296	0.000e+00
3	19.3360	0.2947	0.246	-0.14188	0.000e+00
4	19.6138	0.4008	0.489	-0.14081	0.000e+00
5	19.6248	0.3768	0.498	-0.14079	0.000e+00
6	19.6201	0.3761	0.507	-0.14081	0.000e+00
7	19.6193	0.3673	0.489	-0.14082	0.000e+00
8	19.6201	0.3519	0.489	-0.14083	0.000e+00
9	19.6209	0.3523	0.507	-0.14083	0.000e+00
10	19.6153	0.3523	0.498	-0.14085	0.000e+00
11	19.6169	0.3503	0.507	-0.14084	0.000e+00
12	19.6122	0.3442	0.507	-0.14086	0.000e+00
13	19.6106	0.3443	0.507	-0.14087	0.000e+00
14	19.6122	0.3441	0.507	-0.14086	0.000e+00
15	19.6122	0.3445	0.507	-0.14086	0.000e+00
16	19.6153	0.3446	0.507	-0.14085	0.000e+00
17	19.6146	0.3457	0.507	-0.14086	0.000e+00
18	19.6146	0.3463	0.507	-0.14085	0.000e+00
19	19.6130	0.3469	0.516	-0.14086	0.000e+00
20	19.6074	0.3374	0.570	37.60287	0.000e+00
21	19.6043	0.3420	0.597	20.02945	0.000e+00
22	19.6082	0.3378	0.597	6.93941	0.000e+00
23	19.6074	0.3387	0.624	-4.73907	0.000e+00
24	19.6082	0.3377	0.624	-16.77677	0.000e+00
25	19.6130	0.3378	0.633	-3.97828	0.000e+00
26	19.6138	0.3383	0.633	-0.62769	0.000e+00
27	19.6146	0.3376	0.624	0.04049	0.000e+00
28	19.6177	0.3362	0.633	0.22212	0.000e+00
29	19.6193	0.3374	0.633	0.27583	0.000e+00
30	19.6130	0.3389	0.633	0.32756	0.000e+00
31	19.6043	0.3364	0.633	0.38237	0.000e+00
32	19.6027	0.3356	0.624	0.42163	0.000e+00
33	19.6051	0.3355	0.633	0.43823	0.000e+00
34	19.6027	0.3352	0.633	0.40520	0.000e+00
35	19.6003	0.3354	0.633	0.37638	0.000e+00
36	19.5988	0.3355	0.633	0.35573	0.000e+00
37	19.6019	0.3353	0.633	0.33917	0.000e+00
38	19.5972	0.3351	0.633	0.34541	0.000e+00
39	19.6051	0.3351	0.633	0.32888	0.000e+00
40	19.6067	0.3351	0.642	0.32688	0.000e+00
41	19.6043	0.3351	0.633	0.32468	0.000e+00
42	19.6059	0.3350	0.633	0.29994	0.000e+00
43	19.6043	0.3348	0.633	0.28445	0.000e+00

Ambient
(30' Upstream,
outside of eddy)

```
* Sea-Bird SBE 19 Data File:
* FileName = \SBE19\mtv01.HEX
* Software Version 4.233
* Temperature SN = 2678
* Conductivity SN = 2678
* System UpLoad Time = Oct 05 1999 17:16:00
* ds
* SEACAT PROFILER V3.1b SN 2678   10/05/99  17:16:13.118
* strain gauge pressure sensor: S/N = 184559,  range = 100 psia,
tc = -183
* Narrow Range Conductivity
* clk = 32768.547   iop = 176   vmain = 11.5   vlith = 5.0
* mode = PROFILE  ncasts = 6
* sample rate = 1 scan every 2.0 seconds
* minimum raw conductivity frequency for pump turn on = 2727 hertz
* pump delay = 33 seconds
* samples = 440   free = 115645   lwait = 10 msec
* SW1 = C8   battery cutoff = 7.3 volts
* number of voltages sampled = 2
* logdata = NO
* S>
* cast 1  10/05  11:56:12   samples 66 to 110 sample rate = 1 scan
 every 2.0 seconds  stop = switch off
# nquan = 6
# nvalues = 45
# units = metric
# name 0 = scan: scan number
# name 1 = t068: temperature, IPTS-68 [deg C]
# name 2 = sal: salinity, PSS-78 [PSU]
# name 3 = pr: pressure [d' ]
# name 4 = oxMg/L: oxygen [mg/l]
# name 5 = flag:  0.000e+00
# span 0 = 1, 45
# span 1 = 11.2469, 11.7168
# span 2 = 0.0251, 0.0299
# span 3 = -0.196, 0.435
# span 4 = -5.48483, 55.37061
# span 5 = 0.000e+00, 0.000e+00
# interval = seconds: 0.5
# start_time = Oct 05 1999 11:56:12
# bad_flag = -9.990e-29
# sensor 0 = Frequency 0  temperature, 2678, 12 Jun 99
# sensor 1 = Frequency 1  conductivity, 2678, 12 Jun 99, cpcor = -
9.5700e-08
# sensor 2 = Extrnl Volt  0  oxygen, current, 230792, 16 Jun 99
# sensor 3 = Extrnl Volt  1  oxygen, temperature, 230792, 16 Jun 9
9
# sensor 4 = Pressure Voltage, 184559, 15 Jun 99
```

datcnv_date = Oct 05 1999 17:22:07, 4.233
datcnv_in = MTV01.HEX SBE19.CON
datcnv_skipover = 0
file_type = ascii
END

	Temp (C)				
1	11.2630	0.0274	0.336	-0.17606	0.000e+00
2	11.2621	0.0270	0.336	-0.17606	0.000e+00
3	11.2612	0.0266	0.345	-0.17607	0.000e+00
4	11.2603	0.0262	0.345	-0.17607	0.000e+00
5	11.2790	0.0259	0.354	-0.17598	0.000e+00
6	11.2754	0.0257	0.345	-0.17600	0.000e+00
7	11.2612	0.0259	0.309	-0.17607	0.000e+00
8	11.2772	0.0262	0.354	-0.17599	0.000e+00
9	11.2674	0.0257	0.345	-0.17604	0.000e+00
10	11.2567	0.0257	0.354	-0.17609	0.000e+00
11	11.2737	0.0262	0.426	-0.17601	0.000e+00
12	11.2692	0.0263	0.417	-0.17603	0.000e+00
13	11.2630	0.0266	0.435	-0.17606	0.000e+00
14	11.2701	0.0266	0.435	-0.17602	0.000e+00
15	11.2728	0.0270	0.426	-0.17601	0.000e+00
16	11.2745	0.0270	0.408	-0.17600	0.000e+00
17	11.2665	0.0270	0.417	-0.17604	0.000e+00
18	11.2612	0.0271	0.417	-0.17607	0.000e+00
19	11.2585	0.0272	0.408	-0.17608	0.000e+00
20	11.2621	0.0254	0.408	55.37061	0.000e+00
21	11.2630	0.0254	0.417	50.35672	0.000e+00
22	11.2647	0.0254	0.426	37.36404	0.000e+00
23	11.2674	0.0253	0.435	18.70280	0.000e+00
24	11.2612	0.0254	0.417	-5.48483	0.000e+00
25	11.2576	0.0254	0.417	4.18558	0.000e+00
26	11.2478	0.0256	0.426	9.18046	0.000e+00
27	11.2514	0.0254	0.426	11.78544	0.000e+00
28	11.2567	0.0254	0.417	12.11961	0.000e+00
29	11.2496	0.0253	0.426	12.21503	0.000e+00
30	11.2505	0.0252	0.426	12.05850	0.000e+00
31	11.2523	0.0253	0.435	11.81044	0.000e+00
32	11.2523	0.0254	0.426	11.89470	0.000e+00
33	11.2469	0.0252	0.426	11.77371	0.000e+00
34	11.2496	0.0257	0.435	11.62116	0.000e+00
35	11.2540	0.0251	0.435	11.51490	0.000e+00
36	11.2540	0.0254	0.426	11.43368	0.000e+00
37	11.2532	0.0254	0.426	11.37877	0.000e+00
38	11.2487	0.0253	0.426	11.35576	0.000e+00
39	11.2540	0.0255	0.426	11.31835	0.000e+00
40	11.2523	0.0255	0.426	11.31003	0.000e+00
41	11.2532	0.0254	0.417	11.30499	0.000e+00
42	11.2532	0.0256	0.264	11.27297	0.000e+00
43	11.2540	0.0262	-0.196	11.23033	0.000e+00

Upstream AMZ
(in Eddy)

```
* Sea-Bird SBE 19 Data File:
* FileName = \SBE19\mtv02.HEX
* Software Version 4.233
* Temperature SN = 2678
* Conductivity SN = 2678
* System UpLoad Time = Oct 05 1999 17:16:03
* cs
* SEACAT PROFILER V3.1b SN 2678   10/05/99  17:16:13.118
* strain gauge pressure sensor: S/N = 184559,  range = 100 psia,
tc = -183
* Narrow Range Conductivity
* clk = 32768.547   iop = 176   vmain = 11.5   vlith = 5.0
* mode = PROFILE  ncasts = 6
* sample rate = 1 scan every 2.0 seconds
* minimum raw conductivity frequency for pump turn on = 2727 hertz
* pump delay = 33 seconds
* samples = 440   free = 115645   lwait = 10 msec
* SW1 = C8   battery cutoff = 7.3 volts
* number of voltages sampled = 2
* logdata = NO
* S>
* cast 2  10/05  12:00:00   samples 111 to 193 sample rate = 1 sca
n every 2.0 seconds  stop = switch off
# nquan = 6
# nvalues = 83
# units = metric
# name 0 = scan: scan number
# name 1 = t068: temperature, IPTS-68 [deg C]
# name 2 = sal: salinity, PSS-78 [PSU]
# name 3 = pr: pressure [db]
# name 4 = oxMg/L: oxygen [mg/l]
# name 5 = flag:  0.000e+00
# span 0 = 1, 83
# span 1 = 11.6728, 11.8930
# span 2 = 0.0394, 0.0476
# span 3 = -0.097, 0.859
# span 4 = -6.88572, 57.50526
# span 5 = 0.000e+00, 0.000e+00
# interval = seconds: 0.5
# start_time = Oct 05 1999 12:00:00
# bad_flag = -9.990e-29
# sensor 0 = Frequency 0  temperature, 2678, 12 Jun 99
# sensor 1 = Frequency 1  conductivity, 2678, 12 Jun 99, cpcor = -
9.5700e-08
# sensor 2 = Extrnl Volt  0  oxygen, current, 230792, 16 Jun 99
# sensor 3 = Extrnl Volt  1  oxygen, temperature, 230792, 16 Jun 9
9
# sensor 4 = Pressure Voltage, 184559, 15 Jun 99
```

```
# datcnv_date = Oct 05 1999 17:20:55, 4.233
# datcnv_in = MTV02.HEX SBE19.CON
# datcnv_skipover = 0
# file_type = ascii
*END*
```

	Temp (c)				
1	11.8859	0.0475	0.390	-0.17300	0.000e+00
2	11.8856	0.0475	0.381	-0.17300	0.000e+00
3	11.8853	0.0475	0.381	-0.17300	0.000e+00
4	11.8851	0.0475	0.381	-0.17301	0.000e+00
5	11.8824	0.0474	0.381	-0.17302	0.000e+00
6	11.8833	0.0474	0.363	-0.17301	0.000e+00
7	11.8833	0.0472	0.372	-0.17301	0.000e+00
8	11.8692	0.0469	0.372	-0.17308	0.000e+00
9	11.8427	0.0464	0.372	-0.17321	0.000e+00
10	11.8223	0.0459	0.381	-0.17331	0.000e+00
11	11.8232	0.0457	0.381	-0.17330	0.000e+00
12	11.8232	0.0461	0.345	-0.17330	0.000e+00
13	11.7826	0.0462	0.300	-0.17350	0.000e+00
14	11.8117	0.0461	0.336	-0.17336	0.000e+00
15	11.8683	0.0460	0.363	-0.17309	0.000e+00
16	11.8656	0.0459	0.363	-0.17310	0.000e+00
17	11.8647	0.0461	0.300	-0.17310	0.000e+00
18	11.8789	0.0461	0.318	-0.17303	0.000e+00
19	11.8868	0.0459	0.318	-0.17300	0.000e+00
20	11.8903	0.0476	0.327	57.50526	0.000e+00
21	11.8930	0.0422	0.318	44.66419	0.000e+00
22	11.8047	0.0424	0.318	32.46619	0.000e+00
23	11.7640	0.0428	0.327	15.24481	0.000e+00
24	11.7481	0.0409	0.318	-6.88572	0.000e+00
25	11.7321	0.0400	0.327	5.17053	0.000e+00
26	11.6958	0.0399	0.327	8.22862	0.000e+00
27	11.7242	0.0434	0.327	9.43698	0.000e+00
28	11.7905	0.0446	0.327	9.97058	0.000e+00
29	11.8276	0.0474	0.327	10.22698	0.000e+00
30	11.8745	0.0475	0.327	10.34811	0.000e+00
31	11.8603	0.0469	0.318	10.39361	0.000e+00
32	11.8435	0.0460	0.318	10.35348	0.000e+00
33	11.8586	0.0460	0.327	10.31285	0.000e+00
34	11.8692	0.0464	0.327	10.30028	0.000e+00
35	11.8762	0.0464	0.327	10.29826	0.000e+00
36	11.8497	0.0462	0.327	10.33440	0.000e+00
37	11.8577	0.0447	0.318	10.33379	0.000e+00
38	11.8559	0.0459	0.318	10.30568	0.000e+00
39	11.8329	0.0458	0.318	10.31703	0.000e+00
40	11.8100	0.0447	0.318	10.31417	0.000e+00
41	11.8055	0.0446	0.327	10.31228	0.000e+00
42	11.8064	0.0445	0.327	10.29597	0.000e+00
43	11.8064	0.0447	0.327	10.29914	0.000e+00

44	11.8073	0.0444	0.327	10.29167	0.000e+00
45	11.8117	0.0445	0.327	10.30858	0.000e+00
46	11.8153	0.0443	0.327	10.31235	0.000e+00
47	11.8197	0.0453	0.327	10.30220	0.000e+00
48	11.8365	0.0453	0.327	10.30232	0.000e+00
49	11.8603	0.0460	0.327	10.31520	0.000e+00
50	11.8577	0.0441	0.327	10.31449	0.000e+00
51	11.7826	0.0435	0.318	10.33927	0.000e+00
52	11.8312	0.0455	0.327	10.27913	0.000e+00
53	11.8126	0.0446	0.318	10.29065	0.000e+00
54	11.8250	0.0421	0.327	10.29682	0.000e+00
55	11.7383	0.0394	0.318	10.30043	0.000e+00
56	11.6932	0.0408	0.318	10.27233	0.000e+00
57	11.6728	0.0395	0.246	10.28250	0.000e+00
58	11.6958	0.0401	0.255	10.30443	0.000e+00
59	11.6994	0.0398	0.300	10.30810	0.000e+00
60	11.7366	0.0425	0.723	10.36161	0.000e+00
61	11.7224	0.0430	0.804	10.36810	0.000e+00
62	11.7277	0.0422	0.822	10.35227	0.000e+00
63	11.7162	0.0416	0.831	10.34125	0.000e+00
64	11.7153	0.0414	0.840	10.31915	0.000e+00
65	11.7197	0.0416	0.840	10.32336	0.000e+00
66	11.7277	0.0418	0.840	10.33065	0.000e+00
67	11.7313	0.0418	0.840	10.36480	0.000e+00
68	11.7313	0.0418	0.840	10.35681	0.000e+00
69	11.7295	0.0415	0.849	10.35646	0.000e+00
70	11.7313	0.0418	0.840	10.36391	0.000e+00
71	11.7339	0.0414	0.840	10.35678	0.000e+00
72	11.7286	0.0413	0.840	10.37679	0.000e+00
73	11.728[illegible]	0.0397	0.849	10.37922	0.000e+00
74	11.7277	0.0396	0.859	10.36102	0.000e+00
75	11.7304	0.0403	0.859	10.35612	0.000e+00
76	11.7313	0.0401	0.849	10.35408	0.000e+00
77	11.7304	0.0404	0.859	10.37424	0.000e+00
78	11.7321	0.0410	0.849	10.38329	0.000e+00
79	11.7348	0.0413	0.849	10.41131	0.000e+00
80	11.7675	0.0430	0.426	10.40425	0.000e+00
81	11.7941	0.0443	0.336	10.43076	0.000e+00
82	11.8020	0.0446	0.255	10.42686	0.000e+00
83	11.8073	0.0449	-0.097	10.44808	0.000e+00

Outfall
Station

```
* Sea-Bird SBE 19 Data File:
* FileName = \SBE19\mtv03.HEX
* Software Version 4.233
* Temperature SN = 2678
* Conductivity SN = 2678
* System UpLoad Time = Oct 05 1999 17:16:07
* ds
* SEACAT PROFILER V3.1b SN 2678   10/05/99  17:16:13.118
* strain gauge pressure sensor: S/N = 184559,  range = 100 psia,
tc = -183
* Narrow Range Conductivity
* clk = 32768.547   iop = 176   vmain = 11.5   vlith = 5.0
* mode = PROFILE  ncasts = 6
* sample rate = 1 scan every 2.0 seconds
* minimum raw conductivity frequency for pump turn on = 2727 hertz
* pump delay = 33 seconds
* samples = 440   free = 115645   lwait = 10 msec
* SW1 = C8   battery cutoff = 7.3 volts
* number of voltages sampled = 2
* logdata = NO
* S>
* cast 3  10/05  12:03:50   samples 194 to 298 sample rate = 1 sca
n every 2.0 seconds  stop = switch off
# nquan = 6
# nvalues = 105
# units = metric
# name 0 = scan: scan number
# name 1 = t068: temperature, IPTS-68 [deg C]
# name 2 = sal: salinity, PSS-78 [PSU]
# name 3 = pr: pressure [db]
# name 4 = oxMg/L: oxygen [mg/l]
# name 5 = flag:  0.000e+00
# span 0 = 1, 105
# span 1 = 11.7878, 12.1731
# span 2 = 0.0391, 0.0649
# span 3 = -0.106, 1.967
# span 4 = -6.53347, 58.00916
# span 5 = 0.000e+00, 0.000e+00
# interval = seconds: 0.5
# start_time = Oct 05 1999 12:03:50
# bad_flag = -9.990e-29
# sensor 0 = Frequency 0  temperature, 2678, 12 Jun 99
# sensor 1 = Frequency 1  conductivity, 2678, 12 Jun 99, cpcor = -
9.5700e-08
# sensor 2 = Extrnl Volt  0  oxygen, current, 230792, 16 Jun 99
# sensor 3 = Extrnl Volt  1  oxygen, temperature, 230792, 16 Jun 9
9
# sensor 4 = Pressure Voltage, 184559, 15 Jun 99
```

```
# datcnv_date = Oct 05 1999 17:21:05, 4.233
# datcnv_in = MTV03.HEX SBE19.CON
# datcnv_skipover = 0
# file_type = ascii
*END*
```

	Temp(C)				
1	11.8744	0.0472	0.417	-0.17306	0.000e+00
2	11.8750	0.0472	0.372	-0.17305	0.000e+00
3	11.8756	0.0472	0.399	-0.17305	0.000e+00
4	11.8762	0.0471	0.372	-0.17305	0.000e+00
5	11.8727	0.0469	0.408	-0.17307	0.000e+00
6	11.8674	0.0468	0.399	-0.17309	0.000e+00
7	11.8682	0.0465	0.390	-0.17309	0.000e+00
8	11.8682	0.0465	0.417	-0.17309	0.000e+00
9	11.8356	0.0463	0.390	-0.17324	0.000e+00
10	11.8241	0.0465	0.408	-0.17330	0.000e+00
11	11.8046	0.0467	0.408	-0.17339	0.000e+00
12	11.8082	0.0467	0.408	-0.17338	0.000e+00
13	11.8179	0.0466	0.408	-0.17333	0.000e+00
14	11.8788	0.0465	0.417	-0.17304	0.000e+00
15	11.8744	0.0465	0.417	-0.17306	0.000e+00
16	11.8497	0.0465	0.408	-0.17318	0.000e+00
17	11.8444	0.0465	0.408	-0.17320	0.000e+00
18	11.8453	0.0467	0.408	-0.17320	0.000e+00
19	11.8303	0.0465	0.408	-0.17327	0.000e+00
20	11.8258	0.0463	0.408	58.00916	0.000e+00
21	11.8364	0.0490	0.417	41.49201	0.000e+00
22	11.8594	0.0512	0.417	30.14261	0.000e+00
23	11.8682	0.0485	0.417	14.20154	0.000e+00
24	11.8744	0.0499	0.417	-6.53347	0.000e+00
25	11.9300	0.0465	0.408	6.91425	0.000e+00
26	11.9389	0.0451	0.408	8.98707	0.000e+00
27	11.8691	0.0445	0.408	9.73130	0.000e+00
28	11.8470	0.0447	0.417	10.03564	0.000e+00
29	11.8444	0.0453	0.408	10.17070	0.000e+00
30	11.8576	0.0457	0.417	10.25404	0.000e+00
31	11.8559	0.0458	0.408	10.28222	0.000e+00
32	11.8568	0.0462	0.417	10.32325	0.000e+00
33	11.8647	0.0468	0.408	10.32004	0.000e+00
34	11.8612	0.0461	0.408	10.34738	0.000e+00
35	11.8559	0.0445	0.408	10.32042	0.000e+00
36	11.8444	0.0442	0.408	10.28966	0.000e+00
37	11.8214	0.0441	0.255	10.28947	0.000e+00
38	11.8091	0.0431	0.381	10.25828	0.000e+00
39	11.7878	0.0441	0.597	10.23347	0.000e+00
40	11.8303	0.0391	0.804	10.21477	0.000e+00
41	11.8373	0.0455	0.813	10.25212	0.000e+00
42	11.8709	0.0467	0.813	10.01530	0.000e+00
43	11.8806	0.0473	0.813	10.04302	0.000e+00

44	11.9300	0.0482	0.804	10.13246	0.000e+00
45	11.9194	0.0482	0.804	10.26084	0.000e+00
46	11.9044	0.0483	0.804	10.40714	0.000e+00
47	11.9089	0.0486	0.813	10.36554	0.000e+00
48	11.9142	0.0485	0.804	10.29680	0.000e+00
49	11.9194	0.0488	0.813	10.26046	0.000e+00
50	11.9071	0.0486	0.804	10.26557	0.000e+00
51	11.8656	0.0473	0.813	10.26958	0.000e+00
52	11.8709	0.0472	0.804	10.25696	0.000e+00
53	11.8815	0.0470	0.813	10.24992	0.000e+00
54	11.8797	0.0467	0.804	10.26381	0.000e+00
55	11.8258	0.0461	0.813	10.29424	0.000e+00
56	11.8135	0.0453	0.804	10.24928	0.000e+00
57	11.7940	0.0447	0.804	10.24125	0.000e+00
58	11.8126	0.0447	0.804	10.24848	0.000e+00
59	11.8135	0.0449	0.804	10.29502	0.000e+00
60	11.8338	0.0454	0.804	10.32556	0.000e+00
61	11.8364	0.0454	0.804	10.33593	0.000e+00
62	11.8409	0.0455	0.696	10.33926	0.000e+00
63	11.9203	0.0498	0.633	10.24946	0.000e+00
64	12.1027	0.0556	0.399	10.24309	0.000e+00
65	12.1511	0.0601	0.219	10.39355	0.000e+00
66	12.1731	0.0631	1.526	10.36519	0.000e+00
67	12.1652	0.0630	1.949	10.27197	0.000e+00
68	12.0755	0.0560	1.949	10.20092	0.000e+00
69	12.0543	0.0554	1.949	10.13420	0.000e+00
70	12.0605	0.0556	1.949	10.12324	0.000e+00
71	12.0790	0.0555	1.949	10.16038	0.000e+00
72	12.0895	0.0554	1.949	10.17919	0.000e+00
73	12.0702	0.0558	1.949	10.21356	0.000e+00
74	12.0614	0.0547	1.949	10.22540	0.000e+00
75	12.0438	0.0531	1.949	10.26033	0.000e+00
76	12.0288	0.0538	1.949	10.20798	0.000e+00
77	12.0288	0.0539	1.949	10.19453	0.000e+00
78	12.0244	0.0529	1.949	10.20913	0.000e+00
79	12.0226	0.0566	1.958	10.19437	0.000e+00
80	12.0147	0.0649	1.949	10.29117	0.000e+00
81	12.0138	0.0549	1.958	10.30467	0.000e+00
82	12.0138	0.0553	1.949	10.36304	0.000e+00
83	12.0332	0.0556	1.967	10.25792	0.000e+00
84	12.0270	0.0548	1.949	10.22043	0.000e+00
85	12.0869	0.0554	1.958	10.20730	0.000e+00
86	12.1054	0.0555	1.949	10.18623	0.000e+00
87	12.0587	0.0550	1.949	10.27951	0.000e+00
88	12.0605	0.0584	1.958	10.26696	0.000e+00
89	12.0534	0.0564	1.958	10.36176	0.000e+00
90	12.0534	0.0541	1.958	10.25852	0.000e+00
91	12.0323	0.0551	1.958	10.22364	0.000e+00

Downstream AMZ

```
* Sea-Bird SBE 19 Data File:
* FileName = \SBE19\mtv04.HEX
* Software Version 4.233
* Temperature SN = 2678
* Conductivity SN = 2678
* System UpLoad Time = Oct 05 1999 17:16:12
* ds
* SEACAT PROFILER V3.1b SN 2678   10/05/99  17:16:13.118
* strain gauge pressure sensor: S/N = 184559,  range = 100 psia,
tc = -183
* Narrow Range Conductivity
* clk = 32768.547   iop = 176   vmain = 11.5   vlith = 5.0
* mode = PROFILE  ncasts = 6
* sample rate = 1 scan every 2.0 seconds
* minimum raw conductivity frequency for pump turn on = 2727 hertz
* pump delay = 33 seconds
* samples = 440   free = 115645   lwait = 10 msec
* SW1 = C8   battery cutoff = 7.3 volts
* number of voltages sampled = 2
* logdata = NO
* S>
* cast 4  10/05  12:09:23   samples 299 to 397 sample rate = 1 sca
n every 2.0 seconds  stop = switch off
# nquan = 6
# nvalues = 99
# units = metric
# name 0 = scan: scan number
# name 1 = t068: temperature, IPTS-68 [deg C]
# name 2 = sal: salinity, PSS-78 [PSU]
# name 3 = pr: pressure [db]
# name 4 = oxMg/L: oxygen [mg/l]
# name 5 = flag:  0.000e+00
# span 0 = 1, 99
# span 1 = 11.6350, 12.3411
# span 2 = 0.0358, 0.0639
# span 3 = 0.102, 0.922
# span 4 = -6.89082, 57.64636
# span 5 = 0.000e+00, 0.000e+00
# interval = seconds: 0.5
# start_time = Oct 05 1999 12:09:23
# bad_flag = -9.990e-29
# sensor 0 = Frequency 0  temperature, 2678, 12 Jun 99
# sensor 1 = Frequency 1  conductivity, 2678, 12 Jun 99, cpcor = -
9.5700e-08
# sensor 2 = Extrnl Volt  0  oxygen, current, 230792, 16 Jun 99
# sensor 3 = Extrnl Volt  1  oxygen, temperature, 230792, 16 Jun 9
9
# sensor 4 = Pressure Voltage, 184559, 15 Jun 99
```

```
# datcnv_date = Oct 05 1999 17:21:19, 4.233
# datcnv_in = MTV04.HEX SBE19.CON
# datcnv_skipover = 0
# file_type = ascii
*END*
```

	Temp (c)				
1	11.8270	0.0441	0.336	-0.17329	0.000e+00
2	11.8588	0.0447	0.327	-0.17313	0.000e+00
3	11.8906	0.0452	0.327	-0.17298	0.000e+00
4	11.9224	0.0457	0.318	-0.17283	0.000e+00
5	11.9621	0.0477	0.327	-0.17263	0.000e+00
6	11.9682	0.0490	0.336	-0.17260	0.000e+00
7	11.9083	0.0489	0.327	-0.17289	0.000e+00
8	11.8827	0.0473	0.327	-0.17302	0.000e+00
9	11.8765	0.0467	0.318	-0.17305	0.000e+00
10	11.7837	0.0464	0.318	-0.17349	0.000e+00
11	11.7032	0.0462	0.246	-0.17388	0.000e+00
12	11.6350	0.0467	0.192	-0.17421	0.000e+00
13	11.6731	0.0420	0.237	-0.17403	0.000e+00
14	11.7793	0.0430	0.255	-0.17352	0.000e+00
15	11.7687	0.0436	0.246	-0.17357	0.000e+00
16	11.6500	0.0434	0.255	-0.17414	0.000e+00
17	11.6881	0.0421	0.255	-0.17396	0.000e+00
18	11.7395	0.0415	0.255	-0.17371	0.000e+00
19	11.7589	0.0416	0.246	-0.17362	0.000e+00
20	11.8023	0.0454	0.246	57.64636	0.000e+00
21	11.7837	0.0431	0.246	42.12441	0.000e+00
22	11.8668	0.0451	0.246	30.23681	0.000e+00
23	11.8765	0.0467	0.246	14.03427	0.000e+00
24	11.9197	0.0463	0.246	-6.89082	0.000e+00
25	11.7934	0.0428	0.246	6.26562	0.000e+00
26	11.8597	0.0470	0.246	8.63644	0.000e+00
27	11.9400	0.0491	0.255	9.46818	0.000e+00
28	11.9947	0.0502	0.246	9.83076	0.000e+00
29	11.9938	0.0508	0.255	9.99883	0.000e+00
30	11.9965	0.0512	0.255	10.09522	0.000e+00
31	11.9850	0.0501	0.246	10.12463	0.000e+00
32	11.9947	0.0506	0.246	10.10317	0.000e+00
33	12.0044	0.0515	0.246	10.11705	0.000e+00
34	12.0167	0.0484	0.201	10.14611	0.000e+00
35	11.8994	0.0493	0.417	10.15982	0.000e+00
36	11.9294	0.0481	0.570	10.12168	0.000e+00
37	11.9277	0.0480	0.570	10.23004	0.000e+00
38	11.9206	0.0487	0.570	10.28168	0.000e+00
39	11.9753	0.0495	0.579	10.30607	0.000e+00
40	11.9718	0.0491	0.579	10.27623	0.000e+00
41	11.9744	0.0500	0.570	10.16893	0.000e+00
42	12.0335	0.0531	0.579	10.07172	0.000e+00
43	12.0661	0.0572	0.579	10.17347	0.000e+00

44	12.1162	0.0556	0.579	10.27532	0.000e+00
45	12.0705	0.0524	0.579	10.35009	0.000e+00
46	12.0211	0.0517	0.579	10.32435	0.000e+00
47	12.0158	0.0522	0.579	10.23621	0.000e+00
48	12.0220	0.0512	0.579	10.21550	0.000e+00
49	11.9815	0.0483	0.588	10.22160	0.000e+00
50	11.9735	0.0541	0.579	10.21629	0.000e+00
51	12.1356	0.0626	0.579	10.21601	0.000e+00
52	12.1839	0.0605	0.579	10.33339	0.000e+00
53	12.1672	0.0561	0.579	10.37713	0.000e+00
54	12.1338	0.0523	0.543	10.29480	0.000e+00
55	12.0599	0.0523	0.354	10.20515	0.000e+00
56	12.0247	0.0472	0.606	10.17730	0.000e+00
57	11.9259	0.0461	0.904	10.11584	0.000e+00
58	11.9303	0.0459	0.913	10.04643	0.000e+00
59	12.1892	0.0569	0.913	9.99419	0.000e+00
60	11.9524	0.0420	0.913	10.29884	0.000e+00
61	11.8270	0.0454	0.904	10.26501	0.000e+00
62	11.9418	0.0505	0.913	10.26957	0.000e+00
63	12.1839	0.0596	0.913	10.27766	0.000e+00
64	12.3411	0.0639	0.913	10.35491	0.000e+00
65	12.3034	0.0626	0.913	10.51292	0.000e+00
66	12.2867	0.0604	0.922	10.44613	0.000e+00
67	12.1743	0.0562	0.913	10.33920	0.000e+00
68	11.7598	0.0390	0.922	10.32189	0.000e+00
69	11.6970	0.0406	0.913	10.07608	0.000e+00
70	11.8562	0.0453	0.895	10.07048	0.000e+00
71	11.8482	0.0453	0.904	10.23130	0.000e+00
72	11.8376	0.0456	0.795	10.31453	0.000e+00
73	11.8818	0.0466	0.723	10.39600	0.000e+00
74	11.7996	0.0442	0.777	10.42394	0.000e+00
75	11.7890	0.0428	0.768	10.34118	0.000e+00
76	11.8994	0.0469	0.768	10.31611	0.000e+00
77	11.7890	0.0446	0.759	10.42671	0.000e+00
78	11.9109	0.0459	0.768	10.34462	0.000e+00
79	11.7740	0.0429	0.768	10.44234	0.000e+00
80	11.8288	0.0436	0.768	10.35959	0.000e+00
81	11.8332	0.0454	0.786	10.36762	0.000e+00
82	11.8509	0.0439	0.795	10.43334	0.000e+00
83	11.8226	0.0435	0.795	10.42811	0.000e+00
84	11.7465	0.0371	0.795	10.46337	0.000e+00
85	11.7111	0.0399	0.759	10.37751	0.000e+00
86	11.8482	0.0429	0.750	10.40116	0.000e+00
87	11.8129	0.0436	0.750	10.46787	0.000e+00
88	11.8314	0.0457	0.705	10.49657	0.000e+00
89	11.8694	0.0440	0.480	10.52747	0.000e+00
90	11.7819	0.0436	0.336	10.51229	0.000e+00
91	11.7619	0.0358	0.363	10.45151	0.000e+00

Repeat Upstream Ambient

```
* Sea-Bird SBE 19 Data File:
* FileName = \SBE19\mtv05.HEX
* Software Version 4.233
* Temperature SN = 2678
* Conductivity SN = 2678
* System UpLoad Time = Oct 05 1999 17:16:17
* ds
* SEACAT PROFILER V3.1b SN 2678   10/05/99  17:16:13.118
* strain gauge pressure sensor: S/N = 184559,  range = 100 psia,
tc = -183
* Narrow Range Conductivity
* clk = 32768.547   iop = 176   vmain = 11.5   vlith = 5.0
* mode = PROFILE  ncasts = 6
* sample rate = 1 scan every 2.0 seconds
* minimum raw conductivity frequency for pump turn on = 2727 hertz
* pump delay = 33 seconds
* samples = 440   free = 115645   lwait = 10 msec
* SW1 = C8   battery cutoff = 7.3 volts
* number of voltages sampled = 2
* logdata = NO
* S>
* cast 5  10/05  12:13:41   samples 398 to 439 sample rate = 1 sca
n every 2.0 seconds  stop = switch off
# nquan = 6
# nvalues = 42
# units = metric
# name 0 = scan: scan number
# name 1 = t068: temperature, IPTS-68 [deg C]
# name 2 = sal: salinity, PSS-78 [PSU]
# name 3 = pr: pressure [db]
# name 4 = oxMg/L: oxygen [mg/l]
# name 5 = flag:  0.000e+00
# span 0 = 1, 42
# span 1 = 11.2969, 11.3450
# span 2 = 0.0251, 0.0264
# span 3 = -0.250, 1.093
# span 4 = -6.76392, 59.61066
# span 5 = 0.000e+00, 0.000e+00
# interval = seconds: 0.5
# start_time = Oct 05 1999 12:13:41
# bad_flag = -9.990e-29
# sensor 0 = Frequency 0  temperature, 2678, 12 Jun 99
# sensor 1 = Frequency 1  conductivity, 2678, 12 Jun 99, cpcor = -
9.5700e-08
# sensor 2 = Extrnl Volt  0  oxygen, current, 230792, 16 Jun 99
# sensor 3 = Extrnl Volt  1  oxygen, temperature, 230792, 16 Jun 9
9
# sensor 4 = Pressure Voltage, 184559, 15 Jun 99
```

```
# datcnv_date = Oct 05 1999 17:21:32, 4.233
# datcnv_in = MTV05.HEX SBE19.CON
# datcnv_skipover = 0
# file_type = ascii
*END*
```

	Temp (C)				
1	11.3032	0.0254	0.940	-0.17588	0.000e+00
2	11.3020	0.0255	1.093	-0.17589	0.000e+00
3	11.3008	0.0255	1.084	-0.17589	0.000e+00
4	11.2996	0.0255	1.093	-0.17590	0.000e+00
5	11.2978	0.0254	1.084	-0.17591	0.000e+00
6	11.2996	0.0255	1.093	-0.17590	0.000e+00
7	11.2987	0.0255	1.084	-0.17590	0.000e+00
8	11.2996	0.0254	1.093	-0.17590	0.000e+00
9	11.2996	0.0255	1.093	-0.17590	0.000e+00
10	11.3005	0.0255	1.093	-0.17589	0.000e+00
11	11.3005	0.0254	1.093	-0.17589	0.000e+00
12	11.3005	0.0255	1.093	-0.17589	0.000e+00
13	11.2996	0.0255	1.084	-0.17590	0.000e+00
14	11.2987	0.0255	1.093	-0.17590	0.000e+00
15	11.2978	0.0255	1.084	-0.17591	0.000e+00
16	11.2969	0.0255	1.084	-0.17591	0.000e+00
17	11.2969	0.0256	1.084	-0.17591	0.000e+00
18	11.2978	0.0255	1.084	-0.17591	0.000e+00
19	11.2969	0.0255	1.084	-0.17591	0.000e+00
20	11.2978	0.0254	1.084	59.61066	0.000e+00
21	11.2987	0.0255	1.084	43.69612	0.000e+00
22	11.3005	0.0257	1.084	31.56132	0.000e+00
23	11.2969	0.0254	1.084	14.87854	0.000e+00
24	11.2978	0.0252	1.084	-6.76392	0.000e+00
25	11.2987	0.0258	1.084	6.77195	0.000e+00
26	11.2978	0.0254	1.084	9.32846	0.000e+00
27	11.2969	0.0252	1.084	10.15285	0.000e+00
28	11.2987	0.0254	1.084	10.47029	0.000e+00
29	11.2969	0.0252	1.084	10.61240	0.000e+00
30	11.2996	0.0253	1.084	10.65239	0.000e+00
31	11.2996	0.0256	1.093	10.68680	0.000e+00
32	11.2969	0.0254	1.084	10.72113	0.000e+00
33	11.2978	0.0252	1.093	10.72328	0.000e+00
34	11.2987	0.0255	1.084	10.73520	0.000e+00
35	11.2969	0.0256	1.084	10.72589	0.000e+00
36	11.2969	0.0255	1.084	10.72098	0.000e+00
37	11.2996	0.0252	1.093	10.70381	0.000e+00
38	11.2987	0.0251	1.084	10.68842	0.000e+00
39	11.2987	0.0254	1.093	10.70228	0.000e+00
40	11.2996	0.0254	0.868	10.69926	0.000e+00
41	11.2996	0.0256	-0.124	10.72718	0.000e+00
42	11.3450	0.0264	-0.250	10.61829	0.000e+00

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Appendix D

PLUMES Model Output

Civil, Environmental,

and Recreational

Consulting

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6.38 mgd

Nov 10, 1999, 10: 9: 8 WED PROGRAM PLUMES, Ed 3.1, 8/7/95 Case: 1 of 1

Title Mount Vernon WWTP Outfall nonlinear

tot flow	# ports	port flow	spacing	effl sal	effl temp	far inc	far dis
0.2795	1	0.2795	1000	0.0	19.6	10	100
port dep	port dia	plume dia	total vel	horiz vel	vertl vel	asp coeff	print frq
3.962	0.4572	0.4089	2.128	1.505	1.505	0.10	25
port elev	ver angle	cont coef	effl den	poll conc	decay	Froude #	Roberts F
0.3	45	0.8	-1.65105	100	0	29.56	46610
hor angle	red space	p amb den	p current	far dif	far vel	K:vel/cur	Stratif #
90	1000.0	-0.360913	0.5486	0.0003	0.5486	3.879	0.000
depth	current	density	salinity	temp	amb conc	N (freq)	red grav.
0.0	0.5486	-0.360913	0	11.3		0.000	0.01267
4.0	0.5486	-0.360913	0	11.3		buoy flux	puff-ther
						3.542E-06	5.668
						jet-plume	jet-cross
						11.38	1.406
						plu-cross	jet-strat
						0.02145	
						plu-strat	

hor dis>=

CORMIX1 flow category algorithm is turned off.

45 deg -90 to 90 deg range

Help: F1. Quit: <esc>. Configuration:ATN00. FILE: MTV.VAR;

UM INITIAL DILUTION CALCULATION (nonlinear mode)

plume dep m	plume dia m	poll conc	dilution	hor dis m	
3.962	0.4089	100.0	1.000	0.000	
3.859	0.4766	84.09	1.189	0.1068	
3.752	0.5552	70.71	1.414	0.2252	
3.642	0.6446	59.46	1.681	0.3576	
3.528	0.7454	50.00	1.998	0.5056	
3.412	0.8582	42.05	2.376	0.6711	
3.294	0.9836	35.36	2.826	0.8572	
3.174	1.122	29.73	3.360	1.068	
3.051	1.273	25.00	3.996	1.310	
2.924	1.438	21.02	4.752	1.590	
2.792	1.618	17.68	5.650	1.921	
2.653	1.811	14.87	6.719	2.317	
2.506	2.021	12.50	7.990	2.797	
2.349	2.246	10.51	9.502	3.385	
2.180	2.490	8.839	11.30	4.113	
1.997	2.753	7.433	13.44	5.019	
1.799	3.036	6.250	15.98	6.153	
1.620	3.292	5.403	18.48	7.325	-> surface hit

6.38 mgd

```
Nov 10, 1999,  10: 8:49  WED PROGRAM PLUMES, Ed 3.1, 8/7/95   Case:    1 of    1
Title  Mount Vernon WWTP Outfall                                       nonlinear
 tot flow   # ports port flow   spacing  effl sal effl temp   far inc   far dis
   0.2795         1    0.2795      1000       0.0      19.6        10       100
 port dep  port dia plume dia total vel horiz vel vertl vel asp coeff print frq
    3.962    0.4572    0.4089     2.128     2.128     0.000      0.10        25
port elev ver angle cont coef  effl den poll conc     decay  Froude # Roberts F
      0.3         0       0.8  -1.65105       100         0     29.56     46610
hor angle red space p amb den p current   far dif   far vel K:vel/cur Stratif #
       90    1000.0 -0.360913    0.5486    0.0003    0.5486     3.879     0.000
    depth   current   density  salinity      temp  amb conc  N (freq) red grav.
      0.0    0.5486 -0.360913         0      11.3               0.000   0.01267
      4.0    0.5486 -0.360913         0      11.3           buoy flux puff-ther
                                                            3.542E-06     5.668
                                                            jet-plume jet-cross
                                                                11.38     1.406
                                                            plu-cross jet-strat
                                                              0.02145
                                                            plu-strat

                                                              hor dis>=


CORMIX1 flow category algorithm is turned off.
 -1.65105 sigma-t, 998.349 kg/m3, 0.998349 gm/cm3.   -100 to ~200 sigma-t range
Help: F1.  Quit: <esc>.  Configuration:ATN00.  FILE: MTV.VAR;
UM INITIAL DILUTION CALCULATION (nonlinear mode)
 plume dep plume dia poll conc  dilution   hor dis
         m         m                             m
     3.962    0.4089     100.0     1.000     0.000

     3.962    0.4736     84.09     1.189    0.2038
     3.962    0.5483     70.71     1.414    0.4401
     3.961    0.6011     63.29     1.579    0.6132 -> bottom hit
     3.961    0.6327     59.46     1.681    0.7191 -> bottom hit
     3.960    0.72--     50.00     1.998     1.050 -> bottom hit
     3.959    0.8335     42.05     2.376     1.442 -> bottom hit
     3.957    0.9514     35.36     2.826     1.910 -> bottom hit
     3.954     1.082     29.73     3.360     2.471 -> bottom hit
     3.950     1.226     25.00     3.996     3.148 -> bottom hit
     3.943     1.384     21.02     4.752     3.968 -> bottom hit
     3.934     1.557     17.68     5.650     4.971 -> bottom hit
     3.921     1.745     14.87     6.719     6.204 -> bottom hit
     3.902     1.950     12.50     7.990     7.731 -> bottom hit
     3.875     2.173     10.51     9.502     9.633 -> bottom hit
     3.836     2.415     8.839     11.30     12.02 -> bottom hit
     3.781     2.678     7.433     13.44     15.02 -> bottom hit
     3.703     2.962     6.250     15.98     18.80 -> bottom hit
     3.599     3.270     5.256     19.00     23.39 -> bottom hit
     3.464     3.604     4.420     22.60     28.76 -> bottom hit
     3.298     3.966     3.716     26.87     34.94 -> bottom hit
     3.099     4.359     3.125     31.96     41.97 -> bottom hit
     2.865     4.785     2.628     38.01     49.92 -> bottom hit
     2.606     5.229     2.225     44.88     58.50 -> surface hit -> bottom h
```

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Appendix E

Water Quality-Based Effluent Limit Worksheets

Ammonia
1999 Flows
Existing Outfall

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	8.314
Chronic (n-day) Criteria:	1.877
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.027
Upstream Concentration for Chronic Condition (7Q10):	0.027
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	4.500
Chronic Receiving Water Dilution Factor at 7Q10:	26.000
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
Number of samples (n2) required per month for monitoring:	8

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	37.319
Chronic (n1-day) WLA:	48.127
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	11.982
LTA for Chronic (n1-day) WLA:	25.384
Most Limiting LTA (minimum of acute and chronic):	11.982
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.0440
Daily Maximum Permit Limit:	**37.319**
Monthly Average Permit Limit:	**16.553**

Ammonia
2015 Flows
Existing Outfall

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	8.314
Chronic (n-day) Criteria:	1.877
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.027
Upstream Concentration for Chronic Condition (7Q10):	0.027
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	2.200
Chronic Receiving Water Dilution Factor at 7Q10:	11.200
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
6. Number of samples (n2) required per month for monitoring:	8

Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	18.258
Chronic (n1-day) WLA:	20.747
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	5.862
LTA for Chronic (n1-day) WLA:	10.943
Most Limiting LTA (minimum of acute and chronic):	5.862
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2	0.0440
Daily Maximum Permit Limit:	**18.258**
Monthly Average Permit Limit:	**8.09**

Ammonia
2015 Flows
Existing Outfall

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	8.314
Chronic (n-day) Criteria:	1.877
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.022
Upstream Concentration for Chronic Condition (7Q10):	0.022
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	2.200
Chronic Receiving Water Dilution Factor at 7Q10:	11.200
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
Number of samples (n2) required per month for monitoring:	8

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	18.264
Chronic (n1-day) WLA:	20.798
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	5.864
LTA for Chronic (n1-day) WLA:	10.970
Most Limiting LTA (minimum of acute and chronic):	5.864
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.0440
Daily Maximum Permit Limit:	**18.264**
Monthly Average Permit Limit:	**8.101**

Copper
1999 Flows
Existing Outfall

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	4.610
Chronic (n-day) Criteria:	3.470
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.550
Upstream Concentration for Chronic Condition (7Q10):	0.550
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	4.500
Chronic Receiving Water Dilution Factor at 7Q10:	26.000
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
6. Number of samples (n2) required per month for monitoring:	1

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	18.820
Chronic (n1-day) WLA:	76.470
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	6.043
LTA for Chronic (n1-day) WLA:	40.333
Most Limiting LTA (minimum of acute and chronic):	6.043
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**18.820**
Monthly Average Permit Limit:	**12.901**

Copper
2015 Flows
Existing Outfall

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	4.610
Chronic (n-day) Criteria:	3.470
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.550
Upstream Concentration for Chronic Condition (7Q10):	0.550
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	2.200
Chronic Receiving Water Dilution Factor at 7Q10:	11.200
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
Number of samples (n2) required per month for monitoring:	1

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	9.482
Chronic (n1-day) WLA:	33.254
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	3.045
LTA for Chronic (n1-day) WLA:	17.539
Most Limiting LTA (minimum of acute and chronic):	3.045
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**9.482**
Monthly Average Permit Limit:	**6.500**

Copper
2015 Flows
Extended Outfall

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	4.610
Chronic (n-day) Criteria:	3.470
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.550
Upstream Concentration for Chronic Condition (7Q10):	0.550
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	7.300
Chronic Receiving Water Dilution Factor at 7Q10:	94.800
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
6. Number of samples (n2) required per month for monitoring:	1

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	30.188
Chronic (n1-day) WLA:	277.366
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	9.693
LTA for Chronic (n1-day) WLA:	146.292
Most Limiting LTA (minimum of acute and chronic):	9.693
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**30.188**
Monthly Average Permit Limit:	**20.694**

Lead
Existing Outfall
1999 Flows

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

[redacted]

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	13.900
Chronic (n-day) Criteria:	0.540
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.020
Upstream Concentration for Chronic Condition (7Q10):	0.020
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	4.500
Chronic Receiving Water Dilution Factor at 7Q10:	26.000
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
Number of samples (n2) required per month for monitoring:	1

[redacted]

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	62.480
Chronic (n1-day) WLA:	13.540
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	20.061
LTA for Chronic (n1-day) WLA:	7.141
Most Limiting LTA (minimum of acute and chronic):	7.141
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**22.242**
Monthly Average Permit Limit:	**15.246**

Lead
Existing Outfall
2015 Flows

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	13.900
Chronic (n-day) Criteria:	0.540
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.020
Upstream Concentration for Chronic Condition (7Q10):	0.020
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	2.200
Chronic Receiving Water Dilution Factor at 7Q10:	11.200
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
6. Number of samples (n2) required per month for monitoring:	1

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	30.556
Chronic (n1-day) WLA:	5.844
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	9.811
LTA for Chronic (n1-day) WLA:	3.082
Most Limiting LTA (minimum of acute and chronic):	3.082
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**9.600**
Monthly Average Permit Limit:	**6.581**

Lead
Extended Outfall
2015 Flows

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	13.900
Chronic (n-day) Criteria:	0.540
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.020
Upstream Concentration for Chronic Condition (7Q10):	0.020
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	7.300
Chronic Receiving Water Dilution Factor at 7Q10:	94.800
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
Number of samples (n2) required per month for monitoring:	1

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	101.344
Chronic (n1-day) WLA:	49.316
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	32.540
LTA for Chronic (n1-day) WLA:	26.011
Most Limiting LTA (minimum of acute and chronic):	26.011
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**81.010**
Monthly Average Permit Limit:	**55.531**

Silver
Existing Outfall
1999 Flows

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	0.320
Chronic (n-day) Criteria:	0.320
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.000
Upstream Concentration for Chronic Condition (7Q10):	0.000
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	4.500
Chronic Receiving Water Dilution Factor at 7Q10:	26.000
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
6. Number of samples (n2) required per month for monitoring:	1

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	1.440
Chronic (n1-day) WLA:	8.320
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	0.462
LTA for Chronic (n1-day) WLA:	4.388
Most Limiting LTA (minimum of acute and chronic):	0.462
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**1.440**
Monthly Average Permit Limit:	**0.98[illegible]**

Silver
Existing Outfall
2015 Flows

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	0.320
Chronic (n-day) Criteria:	0.320
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.000
Upstream Concentration for Chronic Condition (7Q10):	0.000
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	2.200
Chronic Receiving Water Dilution Factor at 7Q10:	11.200
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
Number of samples (n2) required per month for monitoring:	1

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	0.704
Chronic (n1-day) WLA:	3.584
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	0.226
LTA for Chronic (n1-day) WLA:	1.890
Most Limiting LTA (minimum of acute and chronic):	0.226
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**0.704**
Monthly Average Permit Limit:	**0.483**

Silver
Extended Outfall
1999 Flows

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	0.320
Chronic (n-day) Criteria:	0.320
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.000
Upstream Concentration for Chronic Condition (7Q10):	0.000
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	13.800
Chronic Receiving Water Dilution Factor at 7Q10:	219.000
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
6. Number of samples (n2) required per month for monitoring:	1

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	4.416
Chronic (n1-day) WLA:	70.080
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	1.418
LTA for Chronic (n1-day) WLA:	36.963
Most Limiting LTA (minimum of acute and chronic):	1.418
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**4.416**
Monthly Average Permit Limit:	**3.02[illegible]**

Silver
Extended Outfall
2015 Flows

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

[redacted]

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	0.320
Chronic (n-day) Criteria:	0.320
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.000
Upstream Concentration for Chronic Condition (7Q10):	0.000
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	7.300
Chronic Receiving Water Dilution Factor at 7Q10:	94.800
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
Number of samples (n2) required per month for monitoring:	1

[redacted]

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	2.336
Chronic (n1-day) WLA:	30.336
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	0.750
LTA for Chronic (n1-day) WLA:	16.000
Most Limiting LTA (minimum of acute and chronic):	0.750
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**2.336**
Monthly Average Permit Limit:	**1.601**

Existing Outfall
1999 Flows

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	35.400
Chronic (n-day) Criteria:	32.300
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	9.300
Upstream Concentration for Chronic Condition (7Q10):	9.300
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	4.500
Chronic Receiving Water Dilution Factor at 7Q10:	26.000
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
6. Number of samples (n2) required per month for monitoring:	1

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	126.750
Chronic (n1-day) WLA:	607.300
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	40.697
LTA for Chronic (n1-day) WLA:	320.310
Most Limiting LTA (minimum of acute and chronic):	40.697
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**126.750**
Monthly Average Permit Limit:	**86.88**

Zinc
Existing Outfall
2015 Flows

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

[redacted]

1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	35.400
Chronic (n-day) Criteria:	32.300
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	9.300
Upstream Concentration for Chronic Condition (7Q10):	9.300
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	2.200
Chronic Receiving Water Dilution Factor at 7Q10:	11.200
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
Number of samples (n2) required per month for monitoring:	1

[redacted]

1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	66.720
Chronic (n1-day) WLA:	266.900
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	21.423
LTA for Chronic (n1-day) WLA:	140.772
Most Limiting LTA (minimum of acute and chronic):	21.423
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.3075
Daily Maximum Permit Limit:	**66.720**
Monthly Average Permit Limit:	**45.736**

APPENDIX L
WATERWORLD™ ARTICLE ON MICROTURBINES

WaterWorld™

Serving the Municipal Water/Wastewater Industry

City Uses Microturbines to Generate Electricity from Biogas

By JOHN K. STECKEL JR.

The City of Allentown (PA) Wastewater Treatment Plant has begun a project to reduce energy and operational costs by using microturbine technology to convert biogas into electricity and heat for use in the treatment process.

The 40 mgd Allentown wastewater treatment plant serves all or parts of 14 townships and boroughs in addition to the City of Allentown. The population in the area served is about ,000. The plant's biogas is collected from two primary digesters and one secondary digester. Each digester is 80 feet in diameter and is a floating-cover type, equipped with a gas recirculation system for mixing. The external heating system maintains the primary digesters at about 100°F and the secondary digester at about 80°F.

PPL Spectrum – a subsidiary of PPL Corp. – is working with the city on the project, which has the goal of supplying about 18 percent of the plant's electrical power. Design work began in September 2000 and construction started in November. Equipment was delivered in mid-December. Work continues as of this date. The system is expected to be fully on-line by the end of February 2001.

Samples of the gas produced in the digesters were analyzed to determine composition by volume and BTU content. The gas composition was about 65 percent CH_4 and about 30 percent CO_2. The BTU calculation resulted in a value of about 630 BTU per cubic foot.

Plant personnel take daily readings of the gas volume produced in the digesters. For this project, calendar year 1999 was used as the baseline period. Daily gas production ranged from ,000 cubic feet to 309,200 cubic feet. Annual ction was slightly more than 78.6 million feet, or an average of 215,300 cubic feet per day.

Currently, the treatment plant uses digester gas for heating sludge and for heating several plant buildings. The plant has two large dual-fuel (methane and natural gas) boilers that provide hot water for sludge heating, and three smaller dual-fuel boilers for building heat. When sufficient digester gas is not available to meet the loads, the plant uses natural gas. About 35 percent of the gas produced was used for heating in 1999. The remaining 65 percent was sent to two waste burners on the roof of the digester control building.

During the baseline period, the plant's electrical demand varied from 1,275 kW to 2,325 kW. Values above 1,500 kW occur rarely and for short periods when flows are high due to rain. Annual electricity consumption was 13.7 million kilowatt-hours and cost was $850,000, resulting in a blended average cost of 6.2 cents per kilowatt-hour. During the same period, annual natural gas consumption was 20,750 therms at a cost of $16,600. Average cost of gas was 80 cents per therm.

Microturbine Technology

The generating equipment selected for this application is manufactured by Capstone Turbine Corp. A total of 12 systems are being installed at the treatment plant. Each system is a compact, low-emission power generator capable of providing up to 30 kW of electric power. Each system incorporates a compressor, recuperator, combustor, turbine and permanent magnet generator. The rotating components are mounted on a single shaft, supported by patented air bearings that rotate at up to 96,000 rpm. The generator is cooled by airflow into the gas turbine, thus eliminating the need for liquid cooling. The output of the system is variable frequency AC power.

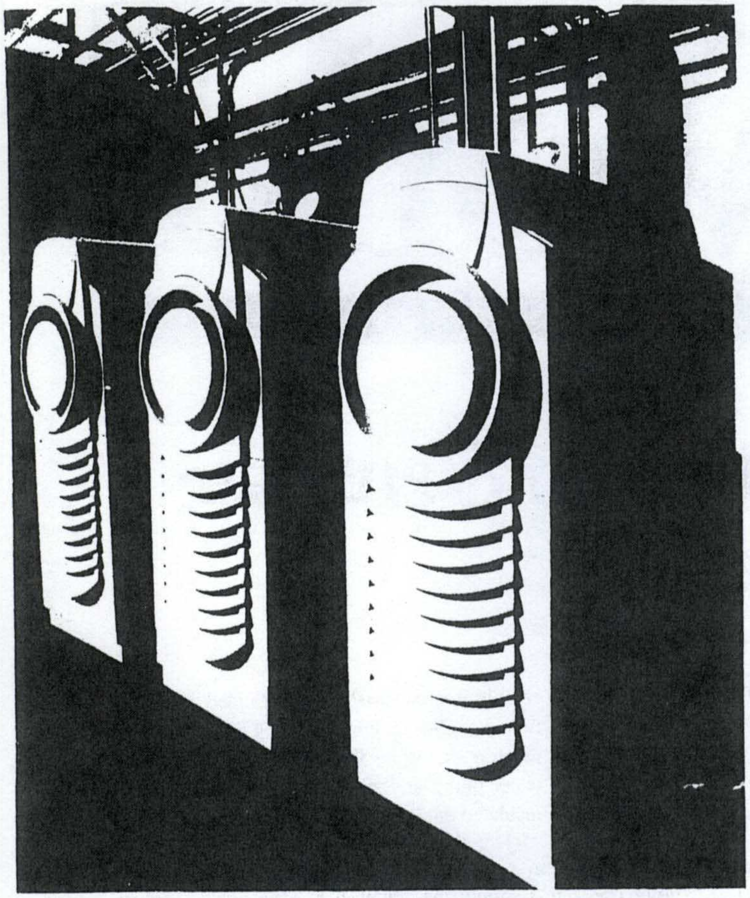
Each microturbine exhausts more than 65 percent of its input energy in a clean, hot gas. A total of three heat recovery systems are being installed. Each system will convert the exhaust energy of four microturbines to hot water. Each heat recovery system consists of an extruded aluminum-finned exchanger core, exchanger enclosure with integral divertor valve for exhaust gas bypass, water pump and associated piping, controls and insulated enclosure.

Hot water from the three heat recovery systems will be piped to existing hot water headers that supply the sludge heat exchangers. Heating loads in the various buildings will continue to be supplied by the dual fuel boilers.

The gas collection system captures digester gas at a low pressure. In order to be used by the microturbines, the gas must be compressed. A fuel gas booster system, manufactured by Copeland, has been designed to increase gas pressure for the microturbines. The system is based on Copeland's proprietary Scroll technology.

The microturbines and their associated equipment are located in a new pavilion near the digesters. The pavilion includes an electrical distribution panel that will provide individual circuit breaker protection for each turbine. A duct bank has been constructed from the new pavilion to a pull box on the existing duct bank system. A new feeder has been installed from the turbine pavilion to the plant's main substation. This arrangement will allow the microturbines to displace utility power for any plant loads, up to their capacity.

Since the plant's minimum electric demand of 1,275 kW is well in excess of the microturbines' capacity (nominally 360 kW total for the 12 microturbines to be installed), there is no need for an interconnection arrangement with the local utility to export power to the grid. The microturbines will operate in a grid connect mode, i.e. only when utility power is available. In the event of a blackout, the microturbines will shut down. Since the plant is served by two feeder lines from the local electric utility, once the plant's substation is switched to the alternate supply line, the microturbines will restart and reload.



The City of Allentown Wastewater Treatment Plant plans to install 12 microturbines manufactured by Capstone Turbine Corp. Each unit is capable of converting biogas into 30 kW of electric power.

hot water that will meet about 96 percent of the plant's thermal requirements. The heat output from the microturbine system is more than adequate to meet this load, except on the coldest winter days. Designers expect about 3 percent of the thermal requirement to be met by excess digester gas that cannot be used by the microturbines and about 1 percent of the thermal requirements to be met by natural gas. Heat output from the microturbines will exceed plant needs the vast majority of the time, particularly in the summer period. About 40 percent of the available heat from the system will be wasted and could potentially be of some use.

Microturbine Operation

Daily gas production varies significantly. The system was designed to use as much of the available gas as possible, but operate the microturbines as baseload units. Since the microturbines come in 30 kW building blocks, staff had flexibility to match the number of microturbines to the pattern of gas production at the facility.

The 12 microturbines being installed will use about 191,500 cubic feet of gas per day at full capacity. An analysis shows that this minimum level of gas will be available over 60 percent of the days. On other days, the generation output of the microturbines will be limited by the availability of fuel. Overall use of the gas in the turbines averages about 87 percent. In addition, about 4 percent of the gas will be used in the boilers to meet the thermal needs of the plant.

The loss in microturbine output due to fuel availability is expected to be about 2 percent of the full-load hours. The impact of scheduled and unscheduled maintenance on the various components of the microturbine system is expected to reduce full-load hours by 5 percent per year. Using a net electrical output from the system of 300 kW, designers expect the annual electrical production to be 2.4 million kilowatt-hours, or 18 percent of the plant's needs.

The microturbine system also will produce

Performance contracting

The use of microturbine technology to generate electric power and heat from digester biogas at Allentown has involved a number of technical and financial risks, as well as a significant up-front investment. The city's use of Performance Contracting with PPL Spectrum provided the means to address these issues and allow the project to move forward.

Under the performance contract, PPL Spectrum will assume all performance risks related to the new equipment and guarantee savings from the project. A 10-year municipal lease, with a bond-comparable interest rate, provides the up-front investment. The city includes monthly payments in its operating budget. The reduction in utility costs, guaranteed by PPL Spectrum, provides money for the payments and net savings in the city's operating budget.

PPL Spectrum expects the microturbine project to result in savings for the city of $25,000 per year or $250,000 over the length of the 10-year contract. After the 10-year period, the up-front investment will have been repaid and net savings to the city will increase to $150,000 per year.

Conclusions

Power production from digester gas has been successful at very large waste treatment plants or large regional solids-processing operations that use anaerobic digesters. Typically, modified combustion gas turbines or internal combustion engine/generator sets have been used with fairly sophisticated pretreatment of the digester gas. Because of the costs involved in such a project, recovery of waste gas to produce electricity at a plant the size of Allentown has been neither practical nor cost effective.

Now, with the use of emerging microturbine technology, PPL Spectrum believes power production from digester gas represents a cost-effective alternative that can be used for economic and environmental benefits at many mid-sized municipal waste treatment plant operations.

This project provides some clear benefits to the City of Allentown and its residents. Reduced emissions from the plant's waste gas burners and the local power plant will provide an environmental benefit. The economic value of this project to the city and its sewage treatment customers is driven by the conversion of a no-cost resource, wasted digester gas, into electricity and heat, which both have value.

Electricity costs are second to personnel costs in the plant's operating budget. The opportunity to displace almost 20 percent of the treatment plant's annual electricity consumption – or $165,000 per year – is a substantial economic benefit.

The benefits shown by PPL Spectrum in the City of Allentown project can be achieved at similar sized facilities across the country. The number of microturbines can be varied to match each plant's gas production levels. In addition, the use of performance contracting provides a means to manage the performance and financial risks for the municipality. WW

***About the Author:** As a Principal for PPL Spectrum since 1995, John K. Steckel Jr. is responsible for the delivery of Energy Management Services to the government and health care sectors. Steckel led the development of a team of energy professionals that have proposed over $30 million of energy driven projects. He has developed energy projects for hospitals, colleges & universities, schools, and government entities. These projects have included a wide variety of technologies, including lighting retrofits, HVAC improvements, energy efficient drives, pumps and motors, combined heat and power projects, water conservation measures and energy management systems.*

APPENDIX M
TECHNICAL MEMORNADUM
AERATION BASIN UPGRADE

City of Mount Vernon
Aeration Basin Upgrade
Draft Memorandum

HDR

Date: October 11, 2001

To: Walt Enquist

cc: John Buckley

From: Brad Einfeld
Monika Blassino

Subject: Mount Vernon Wastewater Treatment Plant Aeration Basin Upgrade.

BACKGROUND

Mount Vernon Wastewater Treatment Plant currently operates four aeration basins with coarse bubble diffusers. Three basins are used in the activated sludge secondary treatment process and the fourth for WAS storage. Due to increased energy costs, the City of Mount Vernon will implement changes to the aeration basins, by installing fine bubble diffusers in all aeration tanks. Fine bubble aeration is significantly more effective in oxygen transfer, therefore less energy is needed to provide the wastewater with the required oxygen. The sections below summarize existing and proposed systems as well as findings regarding power savings attained by replacing coarse bubble diffusers with a fine bubble diffuser system.

EXISTING SYSTEM

The existing system contains four basins, including three equal sized smaller basins and one larger basin. The total volume for all four basins is 196,000 CF, when depth of water is 18 feet. A schematic diagram of the aeration basin configuration is included in Appendix A. Currently the basins operate on a coarse bubble diffuser system with a standard oxygen transfer efficiency of approximately 15.1%.

Four 200 hp centrifugal blowers (Lamson model 1257-AD) are available to supply air to the basins. At current loadings, however, typically only one blower is used. Figure 1 shows coarse bubble head loss curve for current conditions. The current operating range for coarse bubble system is 3,000 SCFM to 4,300 SCFM. The operating curve for these blowers was used for power requirement analysis and it can be found in Appendix B.

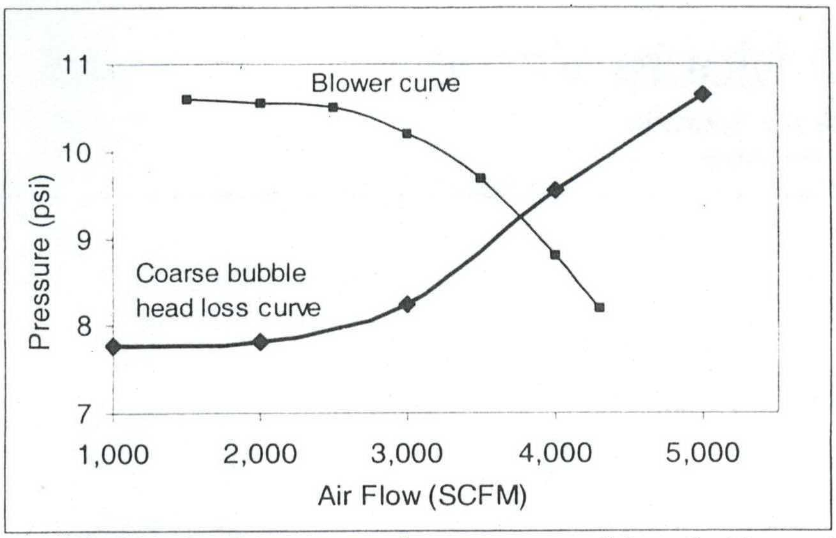


Figure 1: Coarse bubble head loss curve for current conditions (without nitrification).

PROPOSED SYSTEM

The proposed system will utilize the existing basins and blowers and replace the coarse bubble diffusers with fine bubble diffusers. As with coarse bubble diffusers, the fine bubble system will only utilize one blower. Experimental data shows that fine bubble diffusers have standard oxygen transfer efficiency of 37.3%, or up to 2.5 times that of coarse bubble diffusers. For this analysis an increase in efficiency of 2.0 was used. Pertinent information for the fine bubble system obtained from the supplier is provided in Appendix C. Under current conditions, the anticipated blower operating range for air supply for fine bubble diffusers is 1,400 SCFM to 1,700 SCFM. The projected fine bubble head loss curve is shown in Figure 2.

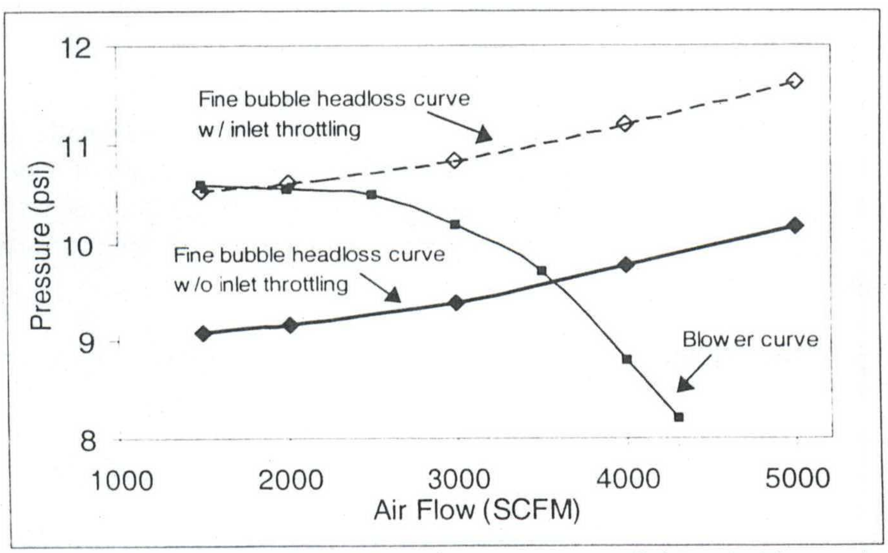


Figure 2: Projected fine bubble head loss curve for current conditions (without nitrification).

As a part of the wastewater treatment plant upgrade, City of Mount Vernon is planning to employ nitrification operations in their system by summer 2002. This upgrade is significant from the point of view of aeration because addition of nitrification to the process will increase the oxygen requirements by as much as 100%. For this analysis an increase in oxygen requirement of 50 % was used. The following analysis shows energy requirements under the existing conditions and conditions with nitrification included.

POWER CONSUMPTION ANALYSIS

To estimate present oxygen demand for the coarse bubble system, hourly power requirements were obtained from Mount Vernon Wastewater Treatment Plant. For this analysis eleven typical days (with average BOD loadings) were chosen from year 2000 operational data. Power requirements were adjusted for blower motor as well as for variable frequency drive (VFD) efficiencies. Operating airflows were determined from manufacturer's blower curves. For the proposed fine bubble system, airflows were reduced by 50% in order to obtain new power requirements. Figure 3 shows existing power requirements for the coarse bubble diffuser system and new requirements for the fine bubble diffuser system. Power consumption is reduced by approximately 30 %, when using fine bubble diffuser system and the conservative assumption of 50 % air demand increase.

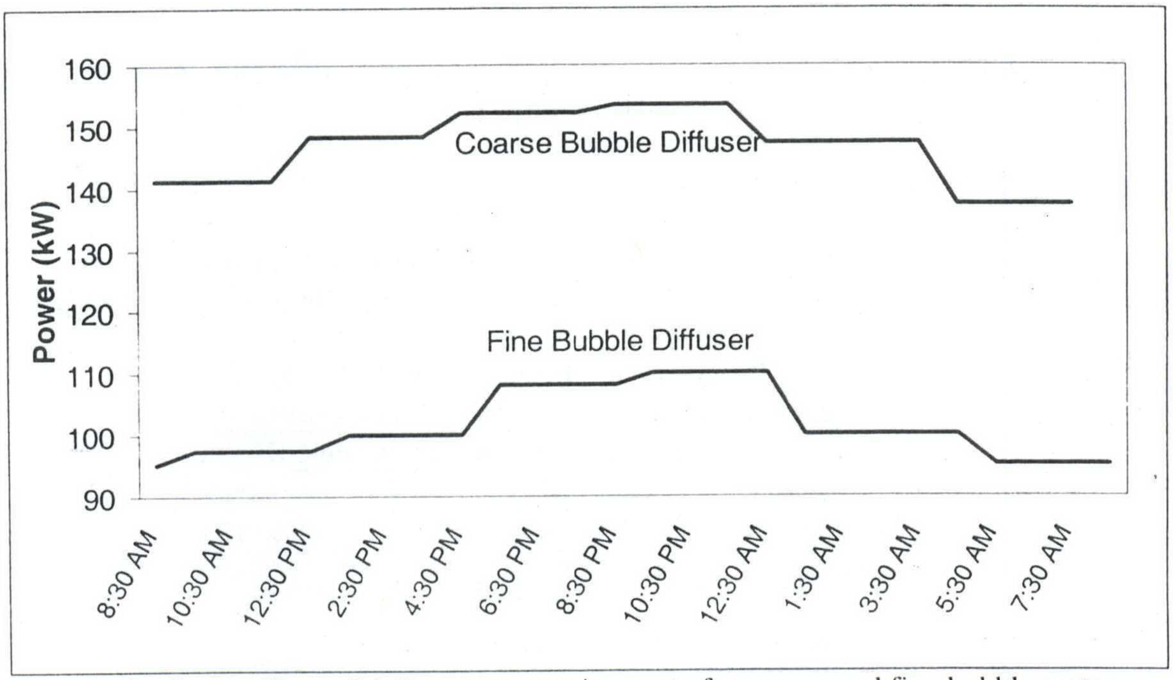


Figure 3: Comparison of daily power requirements for coarse and fine bubble systems (without nitrification).

A similar analysis was performed for a treatment process providing nitrification. To be conservative, a 50% increase in air demand was assumed. It is important to mention that for the coarse bubble system two blowers would have to be utilized to accommodate increase in air demand, whereas fine bubble system would still operate on one blower. Figure 4 shows the daily power requirements for both coarse and fine bubble systems with nitrification. By implementing the fine bubble diffuser system, average power consumption is lowered by 47 %.

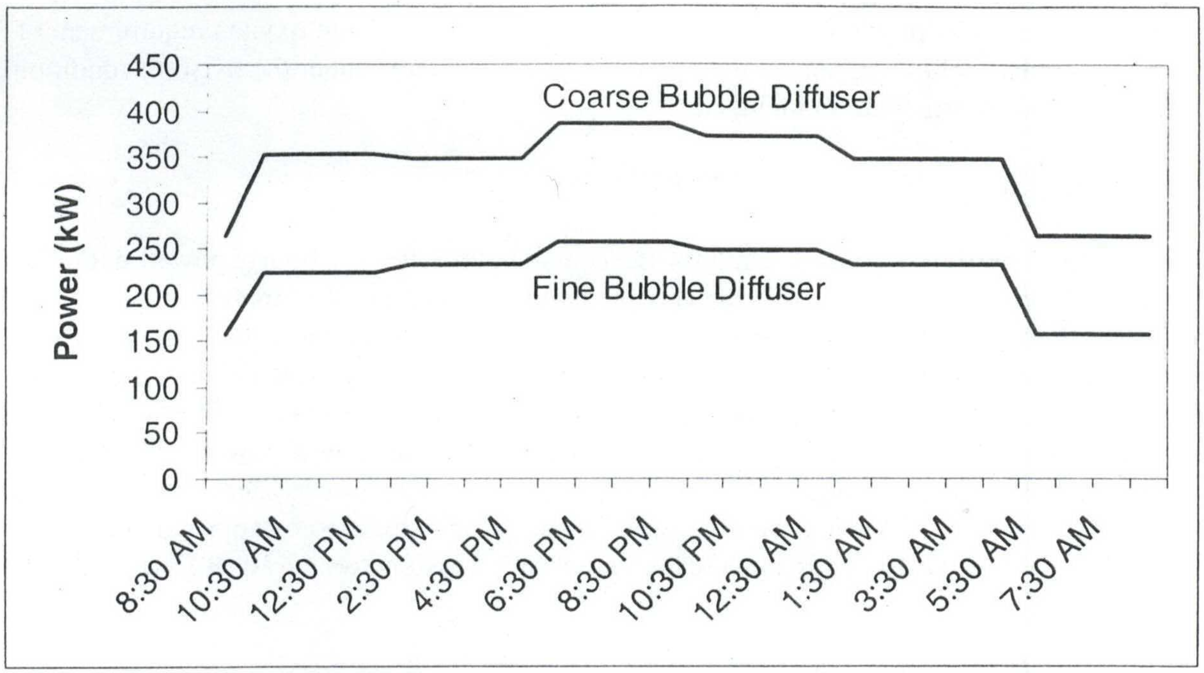


Figure 4: Comparison of daily power requirements for coarse and fine bubble systems (with nitrification).

COST ESTIMATES

Based on existing conditions, by implementing fine bubble aeration, the annual power savings will exceed $23,000. Table 1 shows comparisons of fine and coarse bubble aeration systems.

Table 1: Comparisons of fine and coarse bubble systems for existing aeration basin conditions (no nitrification).

	Coarse Bubble Diffuser	Fine Bubble Diffuser
Existing BOD loading	5,065 lb/d	5,065 lb/d
Power Required	1,284,437 kWh/yr	892,118 kWh/yr
Annual Cost ($0.06/kWh)	$77,070	$53,500
Total Projected Savings per year: $23,570		

After implementation of the nitrification process, the annual power savings will increase to more than $55,000 (Table 2).

Table 2: Comparisons of fine and coarse bubble systems for existing aeration basin conditions (with nitrification).

	Coarse Bubble Diffuser	Fine Bubble Diffuser
Existing BOD loading	5,065 lb/d	5,065 lb/d
Power Required	1,968,810 kWh/yr	1,048,572 kWh/yr
Annual Cost ($0.06/kWh)	$118,128	$62,914
Total Projected Savings per year: $55,214		

Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost (1)	Taxes (8.8%)	Contingency (2) (40%)	Engineering and Administration Costs (3) (25%)	Project Cost (4)
S4-B (7)	8.0	1500	8	LF	$ 177	$ 265,500	$ 23,364	$ 115,546	$ 101,102	**$ 510,000**
S4-B (8)	12.0	1300	8	LF	$ 192	$ 249,600	$ 21,965	$ 108,626	$ 95,048	**$ 480,000**
S4-C (1)	8.0	700	8	LF	$ 177	$ 123,900	$ 10,903	$ 53,921	$ 47,181	**$ 240,000**
S4-C (2)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
S4-C (3)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
S4-C (5)	8.0	700	8	LF	$ 177	$ 123,900	$ 10,903	$ 53,921	$ 47,181	**$ 240,000**
S4-C (4)	8.0	550	8	LF	$ 177	$ 97,350	$ 8,567	$ 42,367	$ 37,071	**$ 190,000**
S4-C (6)	16.0	1250	8	LF	$ 205	$ 256,250	$ 22,550	$ 111,520	$ 97,580	**$ 490,000**
S4-C (7)	18.5	1350	8	LF	$ 177	$ 238,950	$ 21,028	$ 103,991	$ 90,992	**$ 460,000**
S4-C (8)	8.0	950	8	LF	$ 177	$ 168,150	$ 14,797	$ 73,179	$ 64,032	**$ 330,000**
S4-C (9)	8.0	950	8	LF	$ 177	$ 168,150	$ 14,797	$ 73,179	$ 64,032	**$ 330,000**
S3-C (1)	12.0	1300	12	LF	$ 223	$ 289,900	$ 25,511	$ 126,164	$ 110,394	**$ 560,000**
S3-C (2)	11.0	1000	12	LF	$ 223	$ 223,000	$ 19,624	$ 97,050	$ 84,918	**$ 430,000**
S3-C (3)	16.0	900	12	LF	$ 223	$ 200,700	$ 17,662	$ 87,345	$ 76,427	**$ 390,000**
S3-C (4)	8.0	1000	10	LF	$ 203	$ 203,000	$ 17,864	$ 88,346	$ 77,302	**$ 390,000**
S3-C (5)	9.0	1200	12	LF	$ 223	$ 267,600	$ 23,549	$ 116,460	$ 101,902	**$ 510,000**
S3-C (6)	13.8	1100	12	LF	$ 223	$ 245,300	$ 21,586	$ 106,755	$ 93,410	**$ 470,000**
S1-D (1)	8.0	1100	15	LF	$ 270	$ 297,000	$ 26,136	$ 129,254	$ 113,098	**$ 570,000**
S1-D (2)	8.0	800	10	LF	$ 216	$ 172,800	$ 15,206	$ 75,203	$ 65,802	**$ 330,000**
S1-D (3)	8.0	1150	10	LF	$ 216	$ 248,400	$ 21,859	$ 108,104	$ 94,591	**$ 480,000**
PS 1/S4-C & 300 lf FM		770		GPM	$ 780	$ 600,600	$ 52,853	$ 261,381	$ 228,708	**$ 1,150,000**
PS 2/S4-A2 & 1,300 lf FM		150		GPM	$ 5,000	$ 750,000	$ 66,000	$ 326,400	$ 285,600	**$ 1,430,000**
						$ 10,510,000				**$ 20,230,000**
West UGA							$ -	$ -	$ -	
W2-A (1)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
W2-A (2)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
W2-A (3)	8.3	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**

Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost [1]	Taxes (8.8%)	Contingency [2] (40%)	Engineering and Administration Costs [3] (25%)	Project Cost [4]
W2-A (4)	9.3	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	**$ 360,000**
W2-A (5)	10.5	1000	8	LF	$ 192	$ 192,000	$ 16,896	$ 83,558	$ 73,114	**$ 370,000**
W2-A (6)	11.5	1000	8	LF	$ 192	$ 192,000	$ 16,896	$ 83,558	$ 73,114	**$ 370,000**
W2-A (7)	8.3	1100	8	LF	$ 177	$ 194,700	$ 17,134	$ 84,733	$ 74,142	**$ 380,000**
W2-B (1)	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
W2-B (2)	10.0	800	8	LF	$ 185	$ 148,000	$ 13,024	$ 64,410	$ 56,358	**$ 290,000**
W2-B (3)	13.0	800	8	LF	$ 192	$ 153,600	$ 13,517	$ 66,847	$ 58,491	**$ 300,000**
W2-B (4)	16.2	800	8	LF	$ 205	$ 164,000	$ 14,432	$ 71,373	$ 62,451	**$ 320,000**
W2-B (5)	18.2	800	8	LF	$ 205	$ 164,000	$ 14,432	$ 71,373	$ 62,451	**$ 320,000**
W2-B (6)	16.5	500	8	LF	$ 205	$ 102,500	$ 9,020	$ 44,608	$ 39,032	**$ 200,000**
W2-B (7)	13.0	500	8	LF	$ 192	$ 96,000	$ 8,448	$ 41,779	$ 36,557	**$ 190,000**
PS 1/W2-A & 600 lf FM		550		GPM	$ 1,000	$ 550,000	$ 48,400	$ 239,360	$ 209,440	**$ 1,050,000**
						$ 2,820,000				**$ 5,440,000**
East UGA										
E-13	9.0	4500	8	LF	$ 185	$ 832,500	$ 73,260	$ 362,304	$ 317,016	**$ 1,590,000**
E-11 (1)	15.0	3000	8	LF	$ 205	$ 615,000	$ 54,120	$ 267,648	$ 234,192	**$ 1,180,000**
E-11 (2)	8.0	2300	8	LF	$ 177	$ 407,100	$ 35,825	$ 177,170	$ 155,024	**$ 780,000**
E-15	8.0	2500	8	LF	$ 177	$ 442,500	$ 38,940	$ 192,576	$ 168,504	**$ 850,000**
E-14(1)	8.0	2000	8	LF	$ 177	$ 354,000	$ 31,152	$ 154,061	$ 134,803	**$ 680,000**
E-12	8.0	3000	8	LF	$ 177	$ 531,000	$ 46,728	$ 231,091	$ 202,205	**$ 1,020,000**
E-14(2)	8.0	2200	8	LF	$ 177	$ 389,400	$ 34,267	$ 169,467	$ 148,284	**$ 750,000**
E-8 (2)	8.0	1900	8	LF	$ 177	$ 336,300	$ 29,594	$ 146,358	$ 128,063	**$ 650,000**
E-8 (1)	8.0	2300	10	LF	$ 187	$ 430,100	$ 37,849	$ 187,180	$ 163,782	**$ 820,000**
E-8 (3)	8.0	4800	15	LF	$ 237	$ 1,137,600	$ 100,109	$ 495,084	$ 433,198	**$ 2,170,000**
E-8 (4)	13.0	2000	15	LF	$ 255	$ 510,000	$ 44,880	$ 221,952	$ 194,208	**$ 980,000**
E-3 (1)	10.0	1200	8	LF	$ 185	$ 222,000	$ 19,536	$ 96,614	$ 84,538	**$ 430,000**
E-3 (2)	10.0	1200	8	LF	$ 185	$ 222,000	$ 19,536	$ 96,614	$ 84,538	**$ 430,000**

Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost (1)	Taxes (8.8%)	Contingency (2) (40%)	Engineering and Administration Costs (3) (25%)	Project Cost (4)
E-1 (1)	11.3	1000	8	LF	$ 192	$ 192,000	$ 16,896	$ 83,558	$ 73,114	**$ 370,000**
E-1 (2)	10.0	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	**$ 360,000**
E-1 (3)	8.7	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	**$ 360,000**
E-6	8.0	1100	8	LF	$ 177	$ 194,700	$ 17,134	$ 84,733	$ 74,142	**$ 380,000**
E-9 (1)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
E-9 (2)	8.0	1250	8	LF	$ 177	$ 221,250	$ 19,470	$ 96,288	$ 84,252	**$ 430,000**
E-9 (3)	8.0	1250	8	LF	$ 177	$ 221,250	$ 19,470	$ 96,288	$ 84,252	**$ 430,000**
E-7	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
E-4 (1)	8.0	1200	8	LF	$ 177	$ 212,400	$ 18,691	$ 92,436	$ 80,882	**$ 410,000**
E-4 (2)	15.3	1100	8	LF	$ 205	$ 225,500	$ 19,844	$ 98,138	$ 85,870	**$ 430,000**
E-4 (3)	15.3	1000	8	LF	$ 205	$ 205,000	$ 18,040	$ 89,216	$ 78,064	**$ 400,000**
E-4 (4)	13.5	1100	12	LF	$ 238	$ 261,800	$ 23,038	$ 113,935	$ 99,693	**$ 500,000**
E-1 (4)	12.5	2300	15	LF	$ 255	$ 586,500	$ 51,612	$ 255,245	$ 223,339	**$ 1,120,000**
E-1 (5)	11.5	3600	15	LF	$ 255	$ 918,000	$ 80,784	$ 399,514	$ 349,574	**$ 1,750,000**
N10-C (1)	18.0	4000	15	LF	$ 270	$ 1,080,000	$ 95,040	$ 470,016	$ 411,264	**$ 2,060,000**
E-2	8.0	1500	8	LF	$ 177	$ 265,500	$ 23,364	$ 115,546	$ 101,102	**$ 510,000**
N10-H	8.0	1750	8	LF	$ 177	$ 309,750	$ 27,258	$ 134,803	$ 117,953	**$ 590,000**
N10-E	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N10-C (2)	8.0	1400	8	LF	$ 177	$ 247,800	$ 21,806	$ 107,843	$ 94,362	**$ 480,000**
E-5 (1)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
E-5 (2)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
E-5 (3)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
E-10				LF	$ -	$ -	$ -	$ -	$ -	**$ -**
PS 1/East UGA &4,300 lf FM		1770		GPM	$ 1,550	$ 2,743,500	$ 241,428	$ 1,193,971	$ 1,044,725	**$ 5,230,000**
						$ 15,720,000				**$ 30,110,000**

Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost [1]	Taxes (8.8%)	Contingency [2] (40%)	Engineering and Administration Costs [3] (25%)	Project Cost [4]

Notes:

(1) Construction costs include mobilization, excavation, backfill and pipe zone fill, pavement restoration, trench safety, pipe material and installation, manholes, protecting & relocating existing utilities, dewatering, and traffic control.

(2) The construction contingency is an allowance for additional costs not identified in the planning phase and may include utility crossings or unique soil conditions.

(3) Engineering and Administration costs include engineering and design, permitting fees, and City management costs.

(4) Project Costs include taxes (8.8%), contingency (40%), and engineering and administration costs (25%).

APPENDIX D

Related Documents

City of Mount Vernon

Development Services

910 Cleveland Avenue
Post Office Box 809
Mount Vernon, WA 98273

Phone (360) 336-6214
FAX (360) 336-6283
E-Mail DS@ci.mount-vernon.wa.us
www.ci.mount-vernon.wa.us

August 22, 2003

Chris Parsons
Community, Trade and Economic Development
Growth Management Division
P.O. Box 48300
Olympia, Washington 98504-8300

Dear Chris:

I have enclosed for your information an update to **Mount Vernon's Comprehensive Sewer Plan** adding some additional specificity regarding service to the Urban Growth area. We will be taking this to the Planning Commission and City Council for public hearing later in October.

If you have any questions, please give me a call.

Sincerely,

Elizabeth Sjostrom

Elizabeth Sjostróm
Economic Development Planner

Encl.

City of Mount Vernon
Wastewater Division

1401 Britt Road
Mount Vernon, WA 98273-6511

Phone (360) 336-6219
FAX (360) 424-8749
E-Mail mvwwtp@ci.mount-vernon.wa.us
www.ci.mount-vernon.wa.us

August 25, 2003

Bernard Jones
Department of Ecology
Northwest Regional Office
3190 160th Avenue SE
Bellevue, WA 98008-5452

Subject: Notification of sewer line extensions, including pump stations.

Dear Mr. Jones:

In accordance with WAC 173-240-030(5), the City of Mount Vernon is providing notice of planning for sewer pipe extensions, including pump stations in the urban growth area. The sewer expansion areas are shown in Figure 1 attached. The extension plan is in conformance with the Comprehensive Sewer Plan approved by Ecology on March 4, 2003.

If you have any questions on this matter please contact Walt Enquist at (360) 336-6219.

Sincerely,

Walt Enquist

Walt Enquist
Wastewater Utility Supervisor

Attachments

cc: John Buckley, Pubic Works Director
Elizabeth Sjostrom, Economic Development F
NPDES File

F:\Winword\WALT\COMP-PLA\UGA Amendment 2003\DOE Notifica

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AUG 25 2003
Postmark Here
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USPS

Sent To Bernard Jones
Department of Ecology
Street, Apt. No.; or PO Box No. Northwest Regional Office
3190 160th Avenue SE
City, State, ZIP+4 Bellevue, WA 98008-5452

PS Form 3800, April 2002 See Reverse for Instructions

RECOMMENDATIONS

For conditions described above the kWh savings for replacing a coarse bubble system with a fine bubble system would be approximately 920,000 kWh, or about $55,000 annually. Currently, the approximated cost of the installed project is $222,424 (Table 3).

An evaluation was made for a Puget Sound Energy Grant. Based on the analysis above the installed measure cost/annual kWh would be $0.24, and Puget Sound Energy funding of 50% can be anticipated. A completed PSE Grant Application is included in Appendix D.

Table 3: Estimates Cost for Installed System.

Aeration Diffusers (with Sales Tax)	$116,424[1]
Installation (with Sales Tax)	$ 43,000
Engineering, Bidding, Construction Administration	$ 43,000
Contingency (Approximately 10 %)	$ 20,000
	$ 222,424

[1] Based on bid of October 8, 2001.

APPENDIX N
STAFFING CALCULATIONS

By:	Bob Bower, HDR Engineering
Date:	March 15,2003
Plant:	City of Mount Vernon
Location:	Mount Vernon, WA
Version:	2
Type:	Conventional activated sludge with nitrification
Design flow (ADMM):	8.1 mgd in 2010, note 4.

Step 1 - Determine Adjustments for Local Conditions

Category	Discussion	Adjustment (%) Oper.	Maint.	Supv.	Clerical	Lab	Yard
Plant layout	Average						
Unit Processes	Standard Equipment						
Level of Treatment	Secondary						
Removal Requirements	Nitrification/Partial Denit.	5%				10%	
Industrial Wastes	Constant						
Productivity	Average						
Climate	Moderate						
Training	Certification						
SCADA	Monitoring & Control	-5%	5%				
Automatic Sampling	Influent and Effluent	-5%				-5%	
Offsite Lab	Receiving Water Only					-10%	
Offsite Maintenance	Minimal		-5%				
Age of Equipment	50% old, 50% new		5%				
Total		-5%	5%	0%	0%	-5%	0%

Step 2 - Determine Annual Manhours for an Average Plant

Nominal flow	8	mgd
Productivity	1,500	hrs/year

Note 1

Category	Comment		Annual Labor Hours Oper.	Maint.	Supv.	Clerical	Lab	Yard
Supervisory	Note 2				2,100			
Clerical						650		
Laboratory	Lab staff + operators						2,000	
Yardwork								1,850
Raw sewage pumping	Note 2		600	440				
Screening			750	36				
Primary treatment			1,800	460				
Grit removal			625	54				
Primary sludge thickening			320	320				
Aeration			1,400	1,600				
Sec. Clar., RAS, WAS			1,600	390				
UV disinfection	Est. for chlorination		310	390				
WAS thickening			1,200	950				
Anaerobic digestion			775	290				
Dewatering	Note 3	0.4	666					
CSO Outfall O&M				208				
(8hr/day x 4day/wk x 52 wk/yr)	1,664	0.3		499				
Subtotal hours			10,046	5,637	2,100	650	2,000	1,850
Step 3 - Apply Adjustment Factors			-502	282	0	0	-100	0
Total hours	21,962		9,543	5,919	2,100	650	1,900	1,850

Step 4 - Staff Requirement

		Oper.	Maint.	Supv.	Clerical	Lab	Yard
Staff calculated by category	14.6	6.4	3.9	1.4	0.4	1.3	1.2
Staff calculated by total hours	14.6						

Step 5 - Recommendation

Staff requirement	14.5

Reference: **Estimating Staffing for Municipal Wastewater Treatment Plants**, EPA MO-1.

Notes:
1. Excludes sick leave, vacation and holidays (total 29 days) and assumes 6.5 hr/day productive work, 5 days/week
2. Excludes collection system O&M (sewers and lift stations) and time for Engineering, Planning, Construction Management.
3. Dewatering runtime (hr/yr) calculated as shown. Operation labor at 0.4 hr/hr and maintenance at 0.3 hr/hr runtime.
4. Flow from final comp plan. Excludes CSO facilities that are a future phase. This analysis only address through the year 2010.

APPENDIX O
DETERMINATION OF NON-SIGNIFICANCE (DNS)

City of Mount Vernon Community & Economic Development

1002 Cleveland Street
Post Office Box 809
Mount Vernon, WA 98273

Phone (360) 336-6214
FAX (360) 336-6283
E-Mail mvced@ci.mount-vernon.wa.us
www.ci.mount-vernon.wa.us/

DETERMINATION OF NON-SIGNIFICANCE (DNS)

DESCRIPTION OF PROPOSAL: Proposed amendment to the City of Mount Vernon Comprehensive Plan to include an updated Comprehensive Sewer Plan, providing capital facilities planning and needs assessment.

APPLICANT: City of Mount Vernon

LOCATION OF PROPOSAL: City-wide

LEAD AGENCY: Mount Vernon Community and Economic Development Department

The lead agency for this proposal has determined that it does not have a probable adverse impact on the environment. An environmental impact statement (EIS) is not required under RCW 43.21C.030(2)(c). This decision was made after review of a completed environmental checklist and other information on file with the lead agency. This information is available to the public on request.

This DNS is issued under 197-11-340(2); the lead agency will not act on this proposal for 14 days from the date below. Comments must be submitted by December 11, 2000.

RESPONSIBLE PERSON: Rick Cisar
POSITION/TITLE: Community and Economic Development Director
ADDRESS: P. O. Box 809, Mount Vernon, WA 98273
PHONE: (360) 336-6214

RESPONSIBLE PERSON: Rick Cisar
POSITION/TITLE: Community and Economic Development Director
ADDRESS: P. O. Box 809, Mount Vernon, WA 98273
PHONE: (360) 336-6214

DATE 11/14/2000 SIGNATURE Rick Cisar

Published November 27, 2000.
DNS 00-30
Sent to: Ecology, CTED

DETERMINATION OF NON-SIGNIFICANCE (DNS)

DESCRIPTION OF PROPOSAL: Proposed amendment to the City of Mount Vernon Comprehensive Plan to include an updated Comprehensive Sewer Plan, providing capital facilities planning and needs assessment.

APPLICANT: City of Mount Vernon

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RESPONSIBLE PERSON: Rick Cisar
POSITION/TITLE: Community and Economic Development Director
ADDRESS: P. O. Box 809, Mount Vernon, WA 98273
PHONE: (360) 336-6214

Published November 27, 2000.

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City of Mount Vernon

Urban Growth Area Sewer Service Study

October 2003

Prepared by:

HDR

CERTIFICATION PAGE

FOR

City of Mount Vernon
Urban Growth Area Sewer Service Study
Project No. 000 000 000 005237 002

The engineering material and data contained in this Report were prepared under the supervision and direction of the undersigned, whole seal as registered professional engineer is affixed below.

Eric C.M. Bergstrom
Supervising Engineer

City of Mount Vernon

Urban Growth Area Sewer Service Study
Technical Memorandum

Date: October 15, 2003

To: Walt Enquist, Fred Buckenmeyer

From: Eric Bergstrom

Subject: Urban Growth Area Sewer Service Study

INTRODUCTION AND OBJECTIVE

The City is required to establish a plan to provide sewer service to properties within the Urban Growth Area (UGA). This study identifies at a planning level the facilities that would be required to provide sewer service to four major areas within the UGA. These UGA areas are situated to the north, south, east and west of City limits and as illustrated in Figure 1. The areas vary in size from 175 to 1,400 acres.

DRAINAGE BASIN DELINEATION

Each of the UGA service areas shown in Figure 1 was divided into a number of drainage basins. The drainage basins were delineated based on contour and mapping data provided by City of Mt. Vernon geographical information system (GIS) as well as some field reconnaissance. Figures 2, 3, 4, and 5 show the approximate drainage basin boundaries within north, south, east and west UGAs, respectively. Each drainage basin is designated an ID such as E-12, which serves to clarify new sewer locations. The first letter of basin ID refers to a specific UGA, i.e. S for south etc. The basin ID's follow a format set forth in the *City of Mount Vernon Comprehensive Sewer Plan Update*, February 2003.

DESIGN FLOWS

Design flows from each of the drainage basins were estimated based on an assumed population density of 2.5 persons per house and a house density of 4 houses per acre. Average daily flow per capita was assumed to be 100 gallons. Consistent with Comprehensive Sewer Plan, a peaking factor was applied to predict peak daily sanitary flow. The peaking factor for the sanitary flows at any point in the system is based on the following equation:

$$PF = -0.6 \log Q + 2.6$$

where,

PF peaking factor

Q average sanitary flow in million gallons per day (mgd)

Inflow and infiltration (I&I), is independent of sanitary flow and is assumed to be 1,100 gallons per acre per day (gpda). The sanitary sewer peaking factor is not applied to the allowance for I&I.

Table 1 summarizes the drainage basins and calculated average and peak flow for each basin.

Table 1
Drainage Basins Flows

Drainage Basin ID(1)	Basin Area (acres)	Estimated Population(2)	Average Sanitary Flow (gpm)	Peaking Factor	Inflow and Infiltration (gpm)	Peak Design Flow (gpm)
North UGA						
N11-T	26	260	18	3.6	20	84
N15-A	101	1007	70	3.2	77	300
N15-B	132	1320	92	3.1	101	388
N15-C	64	640	44	3.3	49	196
N15-D	59	590	41	3.3	45	182
N16	47	469	33	3.4	36	146
N9-F	25	249	17	3.6	19	80
South UGA						
S1-D	141	1407	98	3.1	107	411
S3-A1	69	689	48	3.3	53	211
S3-A2	167	1672	116	3.1	128	484
S3-B2	84	835	58	3.2	64	252
S3-C	139	1389	96	3.1	106	407
S4-A1	66	661	46	3.3	50	202
S4-A2	99	988	69	3.2	75	295
S4-B	124	1236	86	3.1	94	364
S4-C	274	2737	190	2.9	209	767
S5-B	185	1854	129	3.0	142	533
East UGA						
E-1	209	2088	145	3.0	160	596
E-10	172	1718	119	3.1	131	496
E-11	90	896	62	3.2	68	269
E-12	104	1044	73	3.2	80	311
E-13	129	1286	89	3.1	98	378
E-14	86	861	60	3.2	66	259
E-15	40	400	28	3.4	31	126
E-2	35	347	24	3.5	27	110
E-3	42	416	29	3.4	32	131
E-4	90	903	63	3.2	69	271
E-5	57	575	40	3.3	44	177
E-6	28	285	20	3.5	22	92
E-7	63	633	44	3.3	48	194
E-8	178	1782	124	3.0	136	513
E-9	140	1400	97	3.1	107	410
N10-C	37	375	26	3.5	29	119
N10-E	44	439	30	3.4	34	138
N10-H	55	548	38	3.4	42	169
West UGA						
W1	178	1781	124	3.0	136	513
W2-A	98	977	68	3.2	75	292
W2-B	75	751	52	3.3	57	228

(1) See Figures 2 through 5 for drainage basin designations in each UGA.
(2) Saturated development.

SEWER LAYOUT AND SIZING

Figures 6, 7, 8, and 9 illustrate proposed sewer locations within the UGAs. Where possible, proposed sewers were located in existing right-of-ways. In several locations proposed sewers are shown outside of right-of-way boundaries where these routes provide a more cost effective alignment. Some sewers are shown outside the boundary of the East UGA in order maximize the natural drainage of the area. For this study it was assumed that the sewers outside the UGA boundaries are strictly for conveyance and there are no service connections along the alignment. The piping shown on the figures only includes trunk sewers and interceptors. Small collector sewers that would serve individual properties generally have not been identified.

After proposed alignment of interceptors and major trunk lines was established, grade elevations at approximately 1,000-foot intervals were established based on City's contour mapping. For the purposes of this study it was assumed that the minimum pipe depth to the invert would be 8 feet below grade. In some cases, in order to provide greater slope to improve the flow, greater depths of pipe were assumed. The hydraulic analysis of proposed sewers was completed based on fully developed UGAs for each individual drainage basin. In the model the flows from the drainage basin or portions of the drainage basin are routed into the upstream end of pipe segments.

PROPOSED IMPROVEMENTS

Based on the UGA evaluation proposed sewer improvements were identified. Table 2 summarizes the improvements proposed for each UGA. For detailed list of improvements refer to Appendix A. Figures 6 through 9 illustrate the location of the sewers, forcemains and pump stations and show sewer sizing within each basin.

Table 2
Summary of Proposed Improvements for all UGAs

UGA	Sewer (linear feet)	Pump Stations (ea)	Force Main (linear feet)
North	22,000	3	5,000
South	45,000	2	2,000
East	53,000	1	5,000
West	12,000	1	800

Improvement S3-A2 (1) extends sewer service to intercept flows from basin S4-A2. Initially, sewer service in this basin can be served through SE-A2 (1). Ultimately, as the basin development proceeds, sewer flows will need to be routed down the proposed interceptor on the south side of Maddox Creek and includes sewer segments S4-A1 (1), (2), and (3) so that the sewers on South 19th Street do not become overloaded. The City should monitor flows in the sewer to determine when sewer segments S4-A1 (1), (2), and (3) are required.

Pump station S4-A2 PS1 has been identified at the end of Crosby Drive because the grade to the west may be excessive for a gravity sewer. It may be feasible to extend a gravity sewer to the west and connect to proposed sewer segments S4-A1 (1), (2), and (3) and the City should evaluate the potential for extending gravity service to Crosby Drive as development proceeds.

COST ESTIMATES

Construction Costs

Construction costs for the facilities required to provide sewer service to the Urban Growth Areas were based on cost established in the Comprehensive Sewer Plan Update as well as King County's cost estimating software TABULA. TABULA is a free software developed to estimate costs for sewer projects in the King County area. Use of TABULA allows for cost analysis that is always consistent. The costs can be adjusted to present day values using the ENR cost indexes and/or escalated to future years by using an annual projected inflation multiplier. The inflation multiplier can be defined in the program as any chosen percentage, however, 3 percent is commonly used. An effective 1.13 multiplier was used in the analysis to account for increase in construction cost from 1999 to the end of 2003.

For an example of TABULA's assumptions and item cost breakdown see Appendix B. Note that all assumptions can be defined within the program to better suit a specific project. Table 3 lists costs of pipe, at varying depths, based on TABULA. See Appendix C, column 7, for construction costs for major items associated with each UGA. Costs listed in Table 5 are also construction costs.

The construction costs include excavation and native backfill based on depth and pipe size, standard trench safety, backfill and pipe zone fill, manholes at average spacing of 500 feet, protecting/relocating average complexity utilities, minimal dewatering, traffic control for light traffic conditions, complete pavement restoration for width of trench, and mobilization/demobilization (10 percent of total). These particular costs do not include land acquisition or easements as these elements are difficult to define at the planning stages of design.

Table 3
Construction Cost Estimate Summary Table

Sewer Depth (ft)	Construction Cost per Liner Foot of Sewer								
	8-inch	10-inch	12-inch	15-inch	18-inch	24-inch	27-inch	30-inch	36-inch
8	$177	$187	$207	$237	$258	$316	$341	$385	$452
10	$185	$195	$215	$246	$267	$327	$351	$396	$465
12	$192	$203	$223	$255	$276	$337	$362	$408	$477
15	$205	$216	$238	$270	$292	$355	$381	$429	$500

Notes:
1) Costs are for year 2003.
2) Construction costs include mobilization, excavation, backfill and pipe zone fill, pavement restoration, trench safety, pipe material and installation, manholes, protecting/relocating existing utilities, dewatering, and traffic control.

The costs listed in Table 3 do not include taxes, construction contingency, or engineering and administration costs, which are briefly described below.

Contingency
The construction contingency is an allowance for additional costs not identified in the planning phase. Generally, these costs are not identified because they are unknown at the planning stages of the project. The contingency costs may include complex utility crossings, unique soil conditions, traffic control for heavy traffic conditions, land acquisition, easements, and other unidentified costs. See Appendix C, column 9, for contingency costs on construction costs for major items associated with each UGA.

Engineering and Administration
Engineering and administration costs include engineering and design, permitting process and fees, and City construction management costs. See Appendix C, column 10, for engineering and administration costs on construction costs for major items associated with each UGA.

Project Cost
Project costs include construction costs with an escalation for the following items:

- Sales Tax – 8.8 percent
- Contingency – 40 percent
- Engineering and Administration – 25 percent.

The major construction items and respective costs associated with each UGA are presented in Appendix C. See column 11 of Appendix C, for project costs on construction costs for major items associated with each UGA. Note that Figures 6 through 9 identify individual pipe segments, which are listed in Appendix C. The approximate construction and project costs estimated for each of the UGAs are summarized in Table 4. They include gravity and forcemain piping costs as well as pump station costs.

Table 4
Approximate UGA Construction and Project Costs

Urban Growth Area	Construction Cost (1) (2003)	Project Cost (2) (2003)
North	$6,030,000	$11,580,000
South	$10,510,000	$20,230,000
East	$15,720,000	$30,110,000
East (+ 1,200 ac) (3)	$8,540,000	$16,270,000
East (+ 5,600 ac) (3)	$12,840,000	$24,450,000
West	$2,820,000	$5,440,000

Notes:
(1) Construction costs include mobilization, excavation, backfill and pipe zone fill, pavement restoration, trench safety, pipe material and installation, manholes, protecting & relocating existing utilities, dewatering, and traffic control.
(2) Project costs include sales tax, contingency (unidentified item, such as complex utilities, soil conditions, traffic control, land acquisition, easements, etc.), and engineering and administration costs (engineering and design, permitting, management, etc.).
(3) The expended East service area is described in the proceeding section. Costs for additional 1,200 and 5,600 acres assumes PS1-E, E-8 (4), E-1(4), E-1(5), and N10-C(1) improvements only.

EXPANDED EAST SERVICE AREA

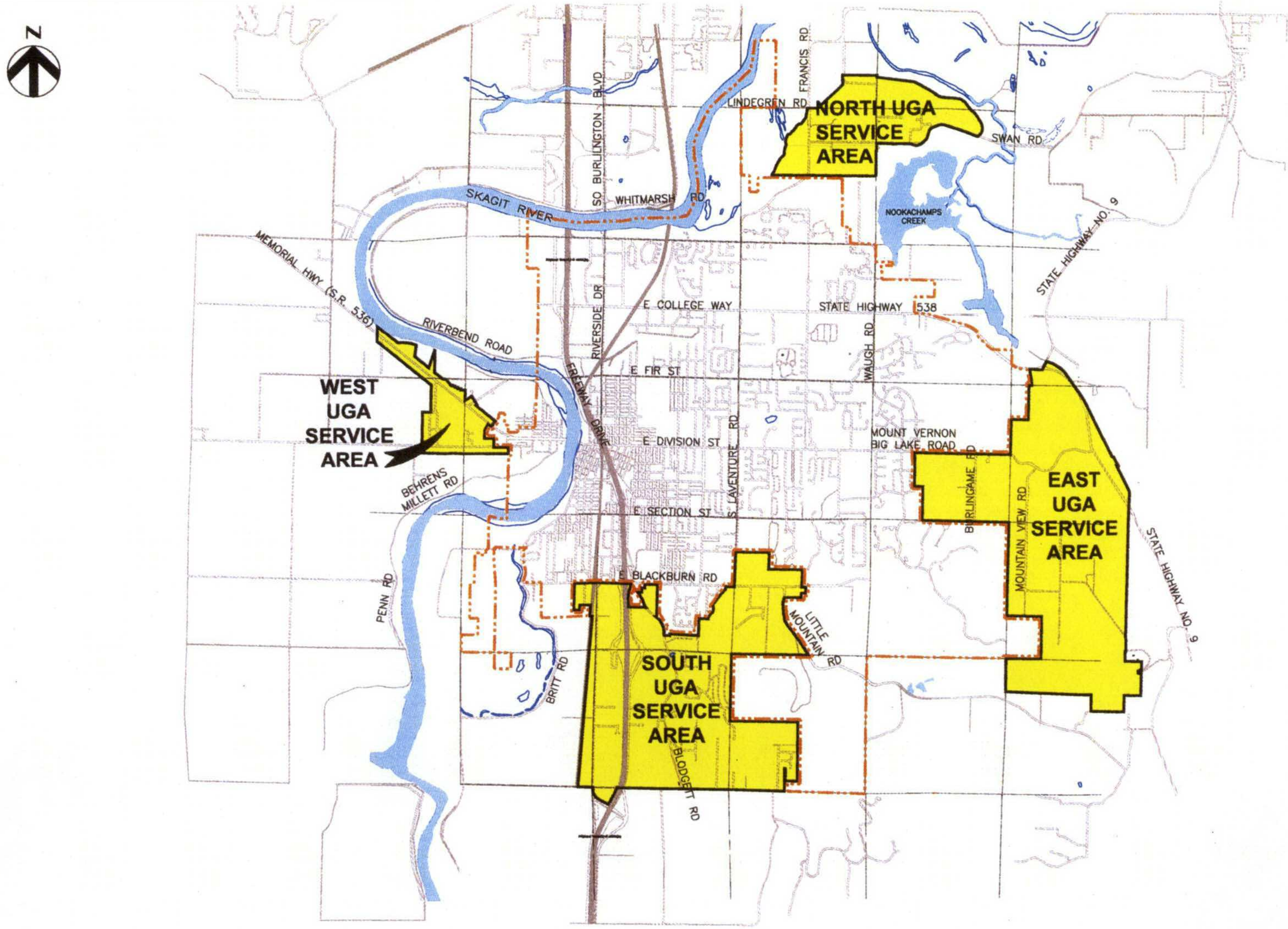
There is potential that sewer service may be provided east of the UGA. The impacts of this service on the sewer sizing within the City was evaluated for different size drainage areas to the east. Table 5 summarizes the impact on the existing sewers within the City along with the sizing of new facilities identified within this study.

Table 5
Expanded Service Area Impacts on Sewer Sizing

Improvement	Sewer Length (linear feet)	Assumed Slope (ft/ft)	Urban Growth Area Service: Design Flows (mgd)	Urban Growth Area Service: Pipe Size (in)	Urban Growth Area Service: Construction Cost [4] (million dollars)	1,200 Additional Acres: Design Flows (mgd)	1,200 Additional Acres: Pipe Size (in)	1,200 Additional Acres: Construction Cost (million dollars)	5,600 Additional Acres: Design Flows (mgd)	5,600 Additional Acres: Pipe Size (in)	5,600 Additional Acres: Construction Cost (million dollars)
PS1 - E	--	--	2.5	--	$2.74 [1]	6.6	--	$4.50 [1]	20.1	--	$7.01 [1]
Forcemain	4,300	--	2.5	12	--	6.6	24[5]	--	20.1	36[5]	--
E-8 (4)	2,000	0.008	2.5	15	$0.51	6.6	24	$0.67	20.1	36	$0.95
E-1 (4)	2,300	0.016	2.7	15	$0.59	7.3	24	$0.78	20.9	36	$1.10
E-1 (5)	3,600	0.007	2.7	15	$0.92	7.3	24	$1.21	20.9	36	$1.72
N10-C (1)	4,000	0.016	4.7	15	$1.08	8.3	24	$1.38	21.8	36	$2.06
East College Way Pump Station	--	--	9.6	--		13.3	--		24.4	--	
East College Way Pump Station Forcemain	6,300	--	9.6 [2]	24		13.3	30		24.4	36	
Kulshan Interceptor	6,860	0.006	18.9 [2]	24 & 30 [3]	--	22.5	30		35.1	36	

Notes:
(1) Cost Includes FM.
(2) Existing flows plus East UGA flows.
(3) Existing piping at capacity is approximately 18 mgd.
(4) Construction costs include mobilization, excavation, backfill and pipe zone fill, pavement restoration, trench safety, pipe material and installation, manholes, protecting/relocating existing utilities, dewatering, and traffic control.
(5) Multiple forcemains could be constructed with this effective pipe size diameter.

Note that costs listed Table 5 are construction costs only. See Appendix B for cost escalations to arrive at project costs.



4000 0 4000 8000

SCALE IN FEET

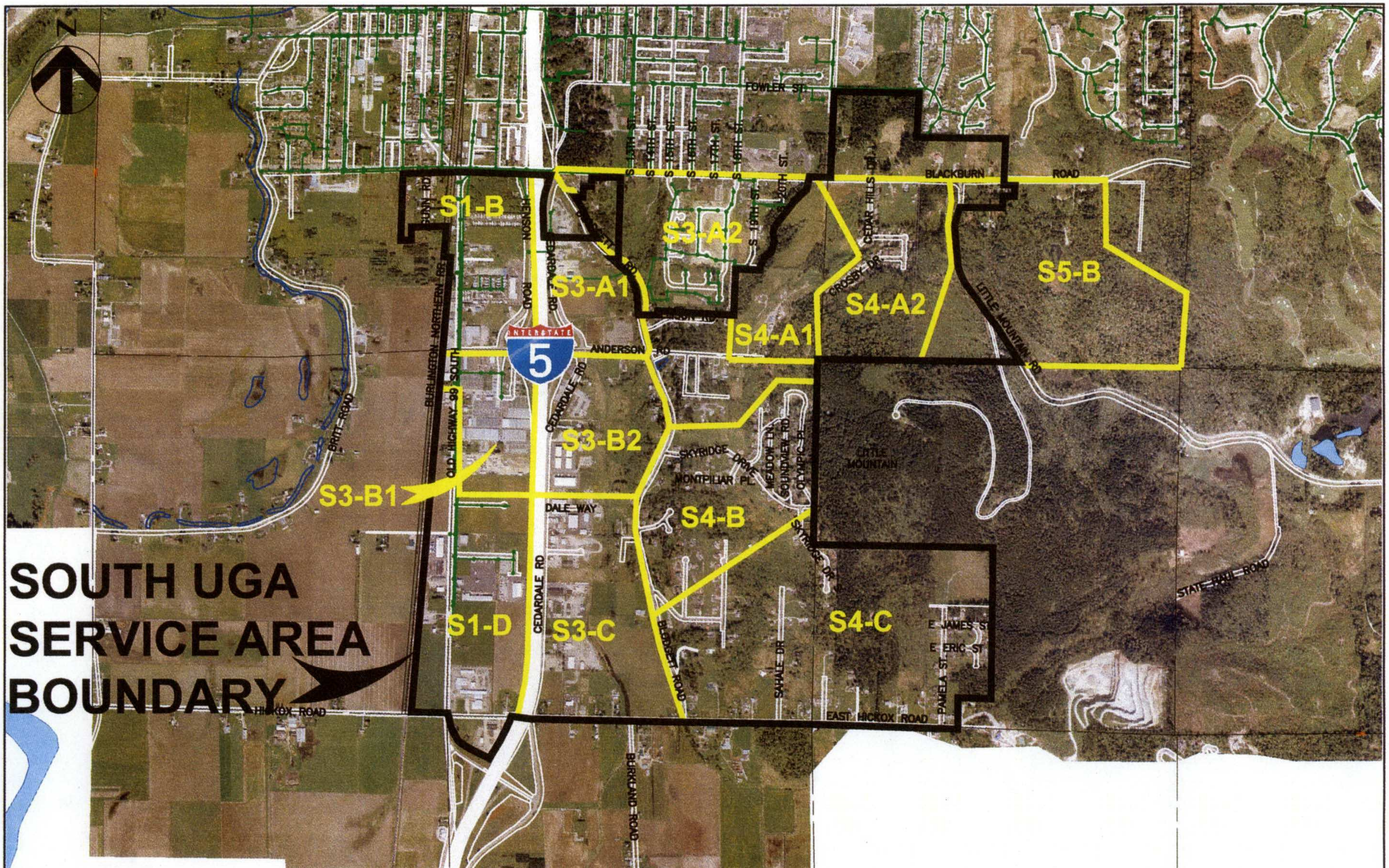
LEGEND

CITY LIMITS

URBAN GROWTH AREA (UGA) EXPANSION

HDR Engineering, Inc.

Project Title	Date
CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA	SEPTEMBER 2003
Sheet Title	Figure No.
URBAN GROWTH AREA (UGA) EXPANSION	1



LEGEND

DRAINAGE BASIN BOUNDARY
EXISTING SEWER

HDR Engineering, Inc.

Project Title: CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA

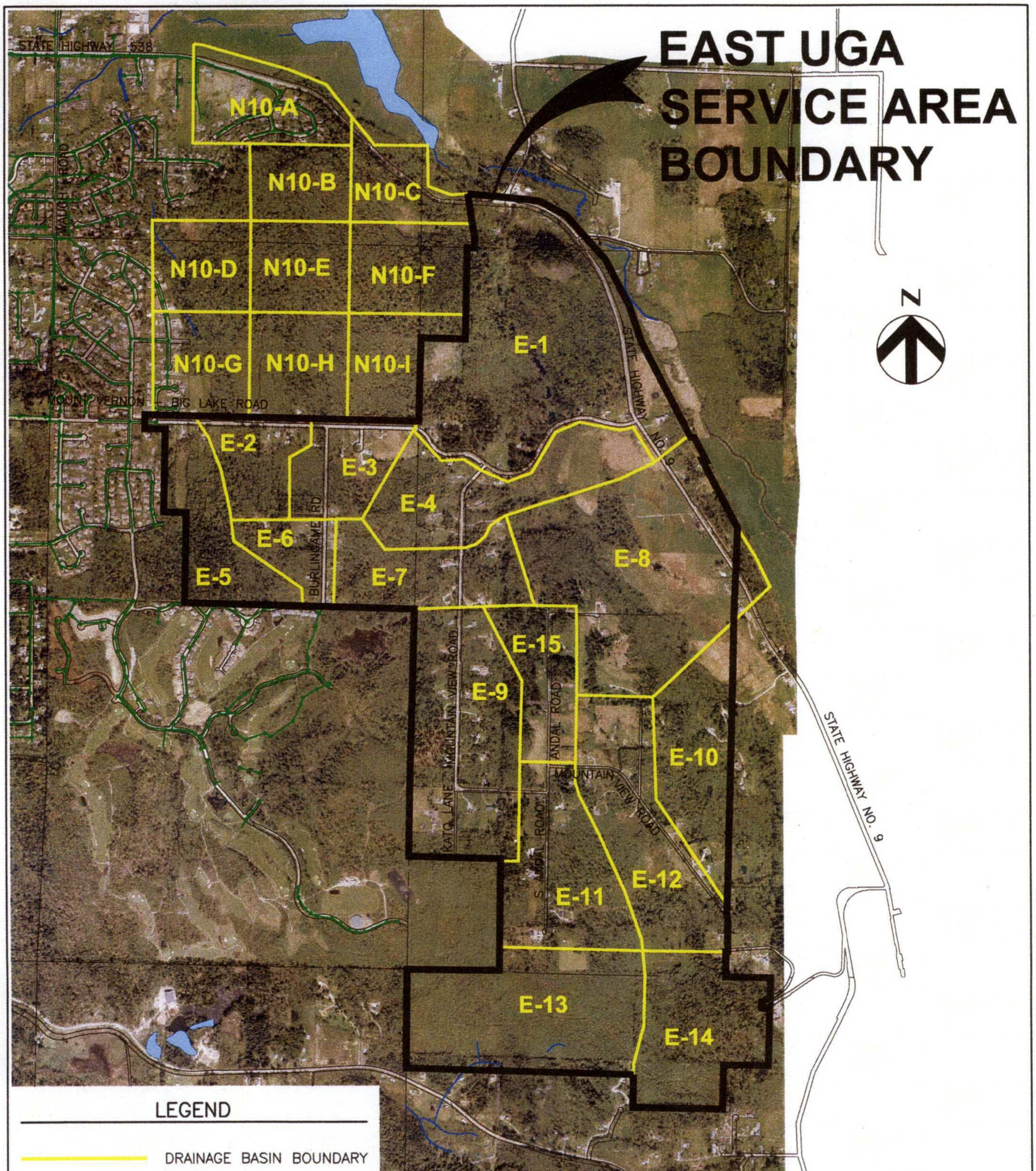
Sheet Title: URBAN GROWTH AREA (UGA) SOUTH SERVICE AREA DRAINAGE BASINS

Figure No. 3

Date: SEPTEMBER 2003

Scale: 1" = 2000'

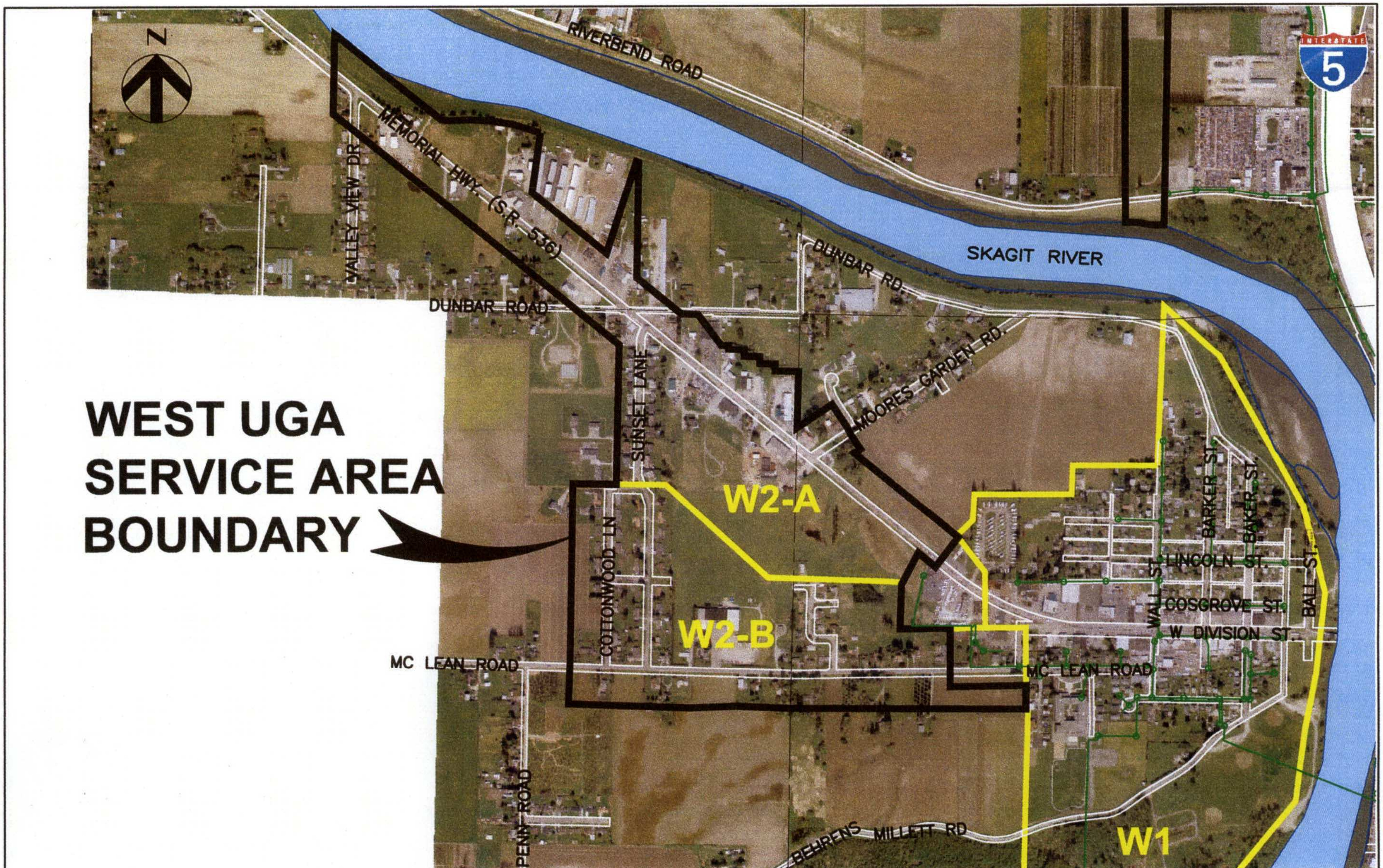
MV-UGA-SOUTH-FIG3.dwg 09/12/03 GUNDY



Project Title: CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA	Figure No. 4	
Sheet Title: URBAN GROWTH AREA (UGA) DRAINAGE BASINS EAST SERVICE AREA	Date AUGUST 2003	Scale 1" = 2000'

HDR Engineering, Inc.

MV-UGA-EAST-FIG5A.dwg 08/20/03 BLUMENFELD



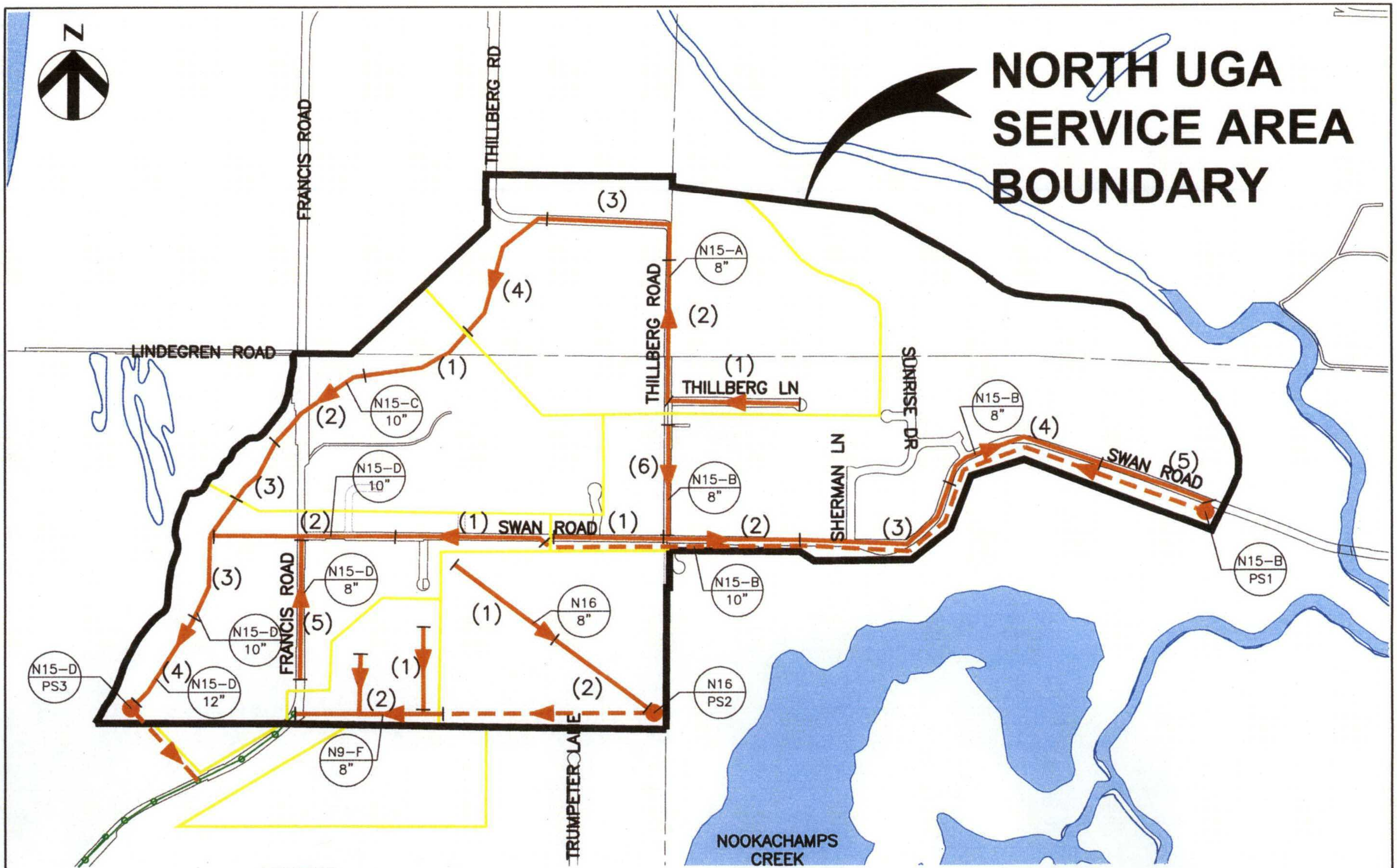
LEGEND

(yellow line)	DRAINAGE BASIN BOUNDARY
(green line)	EXISTING SEWER

HDR Engineering, Inc.

Project Title	CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA	Figure No. 5
Sheet Title	URBAN GROWTH AREA (UGA) WEST SERVICE AREA DRAINAGE BASINS	Date AUGUST 2003
		Scale 1" = 1000'

MV-UGA-WEST-FIG5.dwg 08/20/03 BLUMENFELD

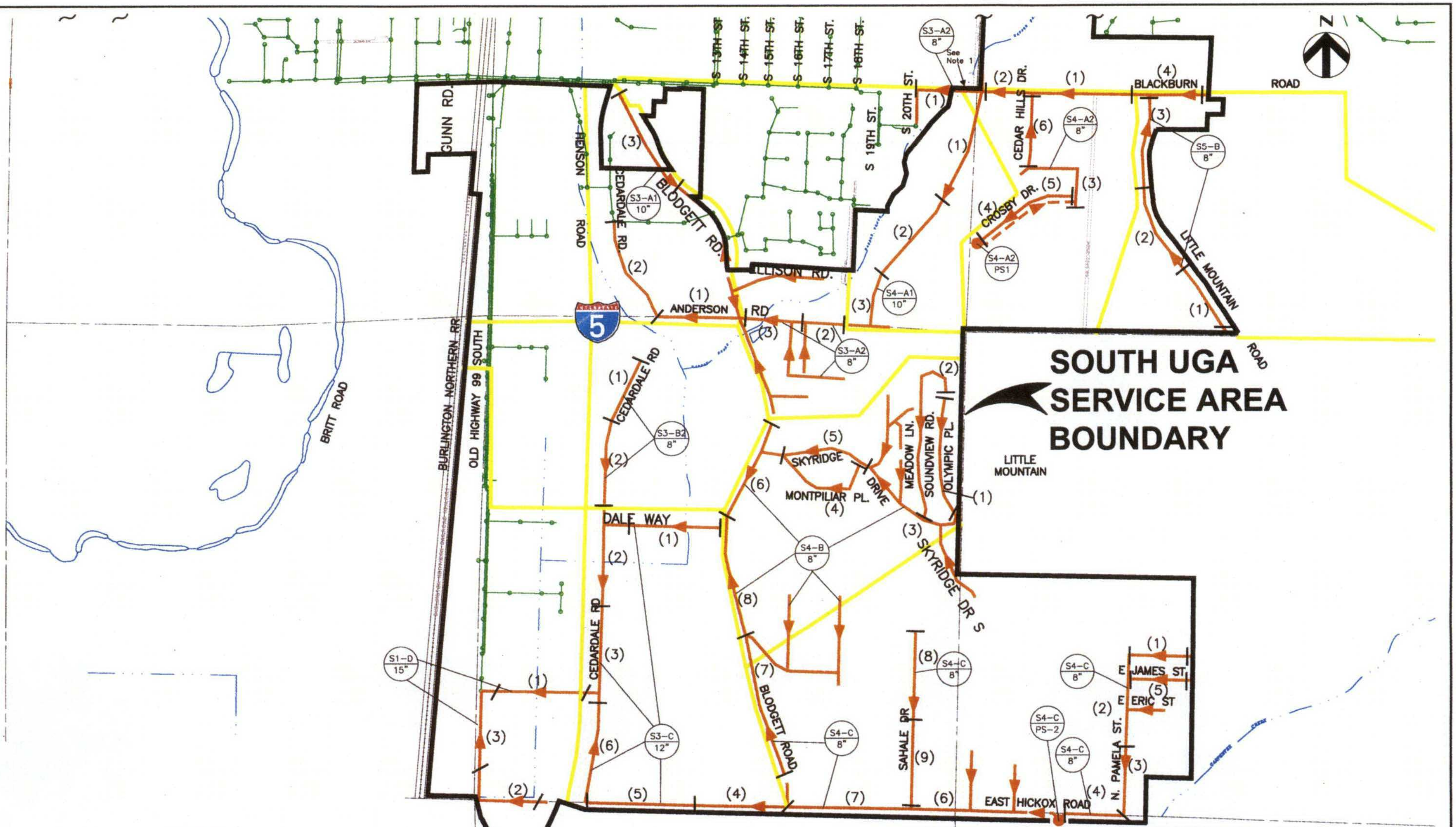


HDR Engineering, Inc.

Project Title	CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA
Sheet Title	URBAN GROWTH AREA (UGA) NORTH SERVICE AREA PROPOSED SEWER PLAN
Figure No.	6
Date	AUGUST 2003
Scale	1" = 1000'

MV–UGA–NORTH–FIG6.dwg   09/08/03 GUNDY

FILENAME: MV-UGA-SOUTH-FIG7.dwg DATE: 09/12/03 GUNDY



LEGEND

- DRAINAGE BASIN BOUNDARY
- EXISTING SEWER
- PROPOSED SEWER
- PROPOSED SEWER FORCEMAIN
- E-3 8" DRAINAGE BASIN ID PIPE SIZE B/W TICKS OR PUMP STA #
- PUMP STATION
- DIRECTION OF FLOW

NOTES:

1. Sewer service to S4-A2 and S5-B may initially be provided via pipe segment S3-A2(1); however, as flows increase in the future, they will need to be diverted to S4-A1 (1), (2), and (3) to prevent overloading. The City should monitor flows to determine when pipe segments S4-A1 (1), (2), and (3) are required.

HDR Engineering, Inc.

Project Title	CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA
Sheet Title	URBAN GROWTH AREA (UGA) SOUTH SERVICE AREA PROPOSED SEWER PLAN
Figure No.	7
Date	SEPTEMBER 2003
Scale	1" = 1000'

LIST OF APPENDICES

APPENDIX A

Hydraulic Analysis Output of the City of Mount Vernon's Urban Growth Area Wastewater Collection System

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
North UGA																		
Thillberg Trunk																		
N15-A (1)	155.0	153.9	147.0	143.0	9.5	1000	8	0.004	101	0.00	0.00	0.111	0.101	3.20	0.43	0.49	87.59	
N15-A (2)	153.9	112.0	143.0	104.0	9.5	1000	8	0.039		0.11	0.10	0.000	0.000	3.20	0.43	1.54	28.05	
N15-A (3)	112.0	85.0	104.0	77.0	8.0	1000	8	0.027		0.11	0.10	0.000	0.000	3.20	0.43	1.28	33.71	
N15-A (4)	85.0	58.0	77.0	50.0	8.0	1300	8	0.0208		0.11	0.10	0.000	0.000	3.20	0.43	1.13	38.44	
N15-C (1)	58.0	57.3	50.0	46.0	9.7	700	10	0.0057	64	0.11	0.10	0.070	0.064	3.07	0.69	1.07	64.15	
N15-C (2)	57.3	56.4	46.0	42.0	12.9	1000	10	0.004		0.18	0.16	0.000	0.000	3.07	0.69	0.90	76.67	
N15-C (3)	56.4	55.8	42.0	40.0	15.1	600	10	0.0033		0.18	0.16	0.000	0.000	3.07	0.69	0.82	83.99	
Swan Road Trunk																		
N15-B (1)	134.1	122.2	126.1	112	9.1	1100	8	0.0128	130	0.00	0.00	0.143	0.130	3.13	0.55	0.88	62.21	
N15-B (2)	122.2	117.5	112	102	12.9	1100	8	0.0091		0.14	0.13	0.000	0.000	3.13	0.55	0.74	73.87	
N15-B (3)	117.5	90.0	102.0	82.0	11.8	1100	8	0.0182		0.14	0.13	0.000	0.000	3.13	0.55	1.05	52.24	
N15-B (4)	90.0	67.6	82.0	59.6	8.0	1000	8	0.0224		0.14	0.13	0.000	0.000	3.13	0.55	1.17	47.06	
N15-B (5)	67.6	57.2	59.6	49.2	8.0	1000	8	0.0104		0.14	0.13	0.000	0.000	3.13	0.55	0.80	69.07	
N15-B (6)	158.7	122.2	150.7	114.2	8.0	800	8	0.0456	2	0.00	0.00	0.002	0.002	4.22	0.01	1.67	0.64	
N15-D (1)	134.1	122.9	126.1	114.9	8.0	1000	10	0.0112	57	0.14	0.13	0.063	0.057	3.04	0.77	1.50	51.62	
N15-D (2)	122.9	44.0	114.9	36.0	8.0	1250	10	0.0631		0.21	0.19	0.000	0.000	3.04	0.77	3.56	21.74	
N15-D (3)	56.0	43.0	40.0	30.0	14.5	1000	10	0.01		0.21	0.19	0.000	0.000	3.04	0.77	1.42	54.63	
N15-D (4)	43.0	42.0	30.0	26.0	14.5	800	12	0.005		0.21	0.19	0.000	0.000	3.04	0.77	1.63	47.51	
N15-D (5)	130.0	105.0	122.0	97.0	8.0	1000	8	0.025	2	0.00	0.00	0.002	0.002	4.22	0.01	1.23	0.86	
N16 (1)	134.7	118.0	126.7	110.0	8.0	1000	8	0.0167	47	0.00	0.00	0.052	0.047	3.40	0.21	1.01	20.89	
N16 (2)	118.0	70.0	110.0	62.0	8.0	800	8	0.06		0.05	0.05	0.000	0.000	3.40	0.21	1.91	11.02	
N9-F (1)	152.0	150.0	144.0	142.0	8.0	600	8	0.0033	25	0.00	0.00	0.028	0.025	3.56	0.31	0.45	68.75	
N9-F (2)	151.0	130.0	143.0	122.0	8.0	1100	8	0.0191		0.08	0.07	0.000	0.000	3.29	0.32	1.08	29.22	
West UGA																		
Dunbar/Sunset Trunk																		
W2-A (1)	23.7	22.3	15.7	14.3	8.0	1000	8	0.0014	47	0.00	0.00	0.052	0.047	3.40	0.21	0.29	72.14	1/2 of W2-A basin
W2-A (2)	22.3	20.9	14.3	12.9	8.0	1000	8	0.0014		0.05	0.05	0.000	0.000	3.40	0.21	0.29	72.32	
W2-A (3)	20.9	19.5	12.9	11.0	8.3	1000	8	0.0019		0.05	0.05	0.000	0.000	3.40	0.21	0.34	62.08	
W2-A (4)	19.5	19.1	11.0	9.1	9.3	1000	8	0.0019		0.05	0.05	0.000	0.000	3.40	0.21	0.34	62.14	1/2 of W2-A basin
W2-A (5)	19.1	18.7	9.1	7.7	10.5	1000	8	0.0014		0.05	0.05	0.000	0.000	3.40	0.21	0.29	73.54	
W2-A (6)	18.7	18.4	7.7	6.4	11.5	1000	8	0.0014		0.05	0.05	0.000	0.000	3.40	0.21	0.29	73.54	
W2-A (7)	20.7	19.9	12.7	11.4	8.3	1100	8	0.0012	2	0.00	0.00	0.002	0.002	4.22	0.01	0.27	3.96	
Cottonwood/Mc Lean Trunk																		

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
W2-B (1)	20.6	17.4	12.6	9.4	8.0	800	8	0.004	36	0.00	0.00	0.040	0.036	3.47	0.16	0.49	33.28	1/2 of W2-B basin
W2-B (2)	17.4	20.2	9.4	8.2	10.0	800	8	0.0015		0.04	0.04	0.000	0.000	3.47	0.16	0.30	54.34	
W2-B (3)	20.2	21.0	8.2	7.0	13.0	800	8	0.0015		0.04	0.04	0.000	0.000	3.47	0.16	0.30	54.34	
W2-B (4)	21.0	23.4	7.0	5.1	16.2	800	8	0.0024		0.04	0.04	0.000	0.000	3.47	0.16	0.38	43.19	
W2-B (5)	23.4	22.6	5.1	4.6	18.2	800	8	0.0006		0.04	0.04	0.000	0.000	3.47	0.16	0.20	84.19	1/2 of W2-B basin
W2-B (6)	22.6	18.3	4.6	3.3	16.5	500	8	0.0026		0.04	0.04	0.000	0.000	3.47	0.16	0.40	41.28	
W2-B (7)	19.0	23.4	11.0	5.4	13.0	500	8	0.0112	2	0.00	0.00	0.002	0.002	4.22	0.01	0.83	1.29	
South UGA																		
Little Mountain/Blackburn Trunk																		
S5-B (1)	372.9	363.0	364.9	355.0	8.0	1000	10	0.0099	123	0.00	0.00	0.135	0.123	3.15	0.52	1.41	37.07	1/3 of S5-B basin
S5-B (2)	363.0	322.5	355.0	314.5	8.0	1000	10	0.0405		0.14	0.12	0.000	0.000	3.15	0.52	2.85	18.33	2/3 of S5-B basin
S5-B (3)	322.5	260.0	314.5	252.0	8.0	990	10	0.0631		0.14	0.12	0.000	0.000	3.15	0.52	3.56	14.68	
S5-B (4)	310.0	260.0	302.0	252.0	8.0	820	10	0.061		0.14	0.12	0.000	0.000	3.15	0.52	3.50	14.94	
S4-A2 (1)	260.0	194.9	252.0	186.9	8.0	1000	10	0.0651	91	0.14	0.12	0.100	0.091	3.00	1.15	3.61	31.83	
S4-A2 (2)	194.9	175.0	186.9	167.0	8.0	600	10	0.0332		0.24	0.21	0.000	0.000	3.00	0.88	2.58	34.04	
S3-A2 (1)	175.0	146.0	167.0	138.0	8.0	1100	10	0.0264	167	0.24	0.21	0.184	0.167	2.85	1.51	2.30	65.51	exist. SS present
Cedar Hills Dr. Trunk																		
S4-A2 (5)	294.0	265.0	286.0	257.0	8.0	550	8	0.0527	4	0.00	0.00	0.004	0.004	4.04	0.02	1.79	1.15	
S4-A2 (4)	265.0	243.0	257.0	235.0	8.0	600	8	0.0367		0.00	0.00	0.000	0.000	4.04	0.02	1.50	1.37	
S4-A2 (3)	294.0	197.0	286.0	189.0	8.0	950	8	0.1021	4	0.00	0.00	0.004	0.004	4.04	0.02	2.50	0.82	
S4-A2 (6)	197.0	194.9	190.0	186.9	7.5	800	8	0.0039		0.00	0.00	0.000	0.000	4.04	0.02	0.49	4.23	
S4-A1 (1)	155.0	150.0	147.0	137.0	10.5	1000	10	0.01	66	0.24	0.21	0.073	0.066	2.93	1.13	1.42	79.75	
S4-A1 (2)	150.0	125.0	137.0	117.0	10.5	1000	10	0.02		0.31	0.28	0.000	0.000	2.93	1.13	2.00	56.39	
S4-A1 (3)	125.0	113.5	117.0	105.5	8.0	1000	10	0.0115		0.31	0.28	0.000	0.000	2.93	1.13	1.52	74.36	
S3-A2 (2)	113.5	80.0	105.5	72.0	8.0	500	8	0.067	167	0.31	0.28	0.184	0.167	2.81	1.75	2.02	86.47	exist. SS present
S3-A2 (3)	80.0	37.6	72.0	29.6	8.0	650	8	0.0652		0.49	0.45	0.000	0.000	2.81	1.75	1.99	87.63	
Blodgett/Anderson Trunk																		
S3-A1 (1)	37.6	20.0	29.6	12.0	8.0	1200	10	0.0147	67	0.31	0.28	0.074	0.067	2.88	1.38	1.71	80.47	
S3-A1 (2)	20.0	18.0	12.0	-1.0	13.5	1000	10	0.013		0.38	0.35	0.000	0.000	2.88	1.38	1.61	85.47	
S3-A1 (3)	65.3	30.0	57.3	22.0	8.0	1700	10	0.0208	2	0.00	0.00	0.002	0.002	4.22	0.01	2.04	0.52	
Cedardale Rd Trunk																		
S3-B2 (1)	21.0	18.0	13.0	10.0	8.0	800	8	0.0038	84	0.00	0.00	0.092	0.084	3.25	0.36	0.48	75.89	
S3-B2 (2)	18.0	16.0	10.0	7.0	8.5	900	8	0.0033		0.09	0.08	0.000	0.000	3.25	0.36	0.45	80.50	
Montpilar/Skyridge Trunk																		
S4-B (1)	340.0	300.0	332.0	292.0	8.0	1400	8	0.0286	124	0.00	0.00	0.136	0.124	3.14	0.52	1.32	39.73	
S4-B (3)	300.0	200.0	292.0	192.0	8.0	1000	8	0.1		0.14	0.12	0.000	0.000	3.14	0.52	2.47	21.24	
S4-B (4)	173.0	96.0	165.0	88.0	8.0	1200	8	0.0642		0.14	0.12	0.000	0.000	3.14	0.52	1.98	26.51	

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
S4-B (5)	173.0	96.0	165.0	88.0	8.0	1100	8	0.07		0.14	0.12	0.000	0.000	3.14	0.52	2.07	25.38	
S4-B (6)	82.0	38.0	74.0	30.0	8.0	1200	8	0.0367		0.14	0.12	0.000	0.000	3.14	0.52	1.50	35.07	
S4-B (7)	50.0	38.3	42.0	30.3	8.0	1500	8	0.0078		0.14	0.12	0.000	0.000	3.14	0.52	0.69	76.04	
S4-B (8)	38.3	38.0	30.3	22.0	12.0	1300	8	0.0064		0.14	0.12	0.000	0.000	3.14	0.52	0.62	84.05	
S4-B (2)	290.0	245.0	282.0	237.0	8.0	1800	8	0.025	2	0.00	0.00	0.002	0.002	4.22	0.01	1.23	0.86	
East Hickox Road/PamelaTrunk																		
S4-C (1)	275.0	225.0	267.0	217.0	8.0	700	8	0.0714	136	0.00	0.00	0.150	0.136	3.12	0.57	2.09	27.49	1/2 of S4-C basin
S4-C (2)	225.0	115.0	217.0	107.0	8.0	1000	8	0.11		0.15	0.14	0.000	0.000	3.12	0.57	2.59	22.16	
S4-C (3)	115.0	65.0	107.0	57.0	8.0	1000	8	0.05		0.15	0.14	0.000	0.000	3.12	0.57	1.75	32.86	
S4-C (4)	65.0	58.0	57.0	50.0	8.0	550	8	0.0127		0.15	0.14	0.000	0.000	3.12	0.57	0.88	65.13	
S4-C (5)	220.0	176.0	212.0	168.0	8.0	700	8	0.0629	1	0.00	0.00	0.001	0.001	4.40	0.01	1.96	0.28	
S4-C (6)	58.0	65.0	50.0	41.0	16.0	1250	8	0.0072	137	0.00	0.00	0.151	0.137	3.12	0.58	0.66	87.20	1/2 of S4-C basin
S4-C (7)	70.0	30.0	41.0	22.0	18.5	1350	8	0.0141		0.15	0.14	0.000	0.000	3.12	0.57	0.93	61.94	
Sahale Drive																		
S4-C (8)	190.0	150.0	182.0	142.0	8.0	950	8	0.0421	2	0.00	0.00	0.002	0.002	4.22	0.01	1.60	0.66	
S4-C (9)	150.0	70.0	142.0	62.0	8.0	950	8	0.0842		0.00	0.00	0.000	0.000	4.22	0.01	2.27	0.47	
Cedardale/Dale Way Trunk																		
S3-C (1)	38.0	16.0	22.0	8.0	12.0	1300	12	0.0108	93	0.23	0.21	0.102	0.093	2.91	1.20	2.39	50.35	2/3 of S3-C basin
S3-C (2)	16.0	14.7	8.0	0.7	11.0	1000	12	0.0073		0.33	0.30	0.000	0.000	2.91	1.20	1.97	61.15	
S3-C (3)	14.7	12.0	0.7	-6.0	16.0	900	12	0.0074		0.33	0.30	0.000	0.000	2.91	1.20	1.99	60.56	
S3-C (4)	30.0	17.0	22.0	9.0	8.0	1000	10	0.013	93	0.15	0.14	0.102	0.093	2.98	0.94	1.61	57.93	1/3 of S3-C basin
S3-C (5)	17.0	13.5	9.0	3.5	9.0	1200	12	0.0046		0.25	0.23	0.000	0.000	2.98	0.94	1.56	59.99	
S3-C (6)	13.5	12.0	3.9	-6.0	13.8	1100	12	0.009		0.25	0.23	0.000	0.000	2.98	0.94	2.18	42.81	
S1-D (1)	12.0		4.0	-8.0	8.0	1100	15	0.005	141	0.48	0.44	0.155	0.141	2.74	2.22	2.95	75.05	no contour data
S1-D (2)					8.0	800	10	0.005	141	0.00	0.00	0.155	0.141	3.11	0.59	1.00	59.29	no contour data
S1-D (3)					8.0	1150	10	0.005		0.16	0.14	0.000	0.000	3.11	0.59	1.00	59.29	no contour data
East UGA																		
E-13	500.0	494.0	492.0	484.0	9.0	4500	8	0.0018	65	0.00	0.00	0.071	0.065	3.31	0.28	0.33	86.46	
Andal Road Trunk																		
E-11 (1)	496.0	476.0	484.0	458.0	15.0	3000	8	0.0087	90	0.07	0.06	0.099	0.090	3.09	0.65	0.73	88.96	
E-11 (2)	541.0	484.0	533.0	476.0	8.0	2300	8	0.0248		0.10	0.09	0.000	0.000	3.23	0.39	1.23	31.68	
E-15	484.0	350.0	476.0	342.0	8.0	2500	8	0.0536	40	0.17	0.15	0.044	0.040	3.03	0.80	1.81	44.39	

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
E-12	484.0	400.0	476.0	392.0	8.0	3000	8	0.028	104	0.00	0.00	0.114	0.104	3.19	0.45	1.31	34.14	
Mountain View Road Trunk																		
E-14(1)	483.0	231.0	475.0	223.0	8.0	2000	8	0.126	86	0.00	0.00	0.095	0.086	3.24	0.37	2.77	13.46	
E-14(2)	231.0	100.0	223.0	92.0	8.0	2200	8	0.0595		0.21	0.19	0.000	0.000	3.03	0.79	1.91	41.20	
State Highway 9 Trunk																		
E-8 (2)	337.0	112.0	329.0	104.0	8.0	1900	8	0.1184	176	0.21	0.19	0.194	0.176	2.86	1.47	2.69	54.57	
E-8 (3)	112.0	100.0	104.0	92.0	8.0	4800	15	0.0025		0.41	0.37	0.000	0.000	2.86	1.47	2.09	70.26	
E-8 (1)	145.0	112.0	137.0	104.0	8.0	2300	10	0.0143	2	0	0.00	0.002	0.002	4.22	0.22	1.70	13.24	
E-8 (4)	163.9	158.0	155.9	140.0	13.0	2000	15	0.008		0.62	0.56	0.000	0.000	2.86	2.22	3.72	59.61	
Burlingame Road / Big Lake Road Trunk																		
E-3 (1)	391.0	393.0	383.0	381.0	10.0	1200	8	0.0017	42	0.00	0.00	0.046	0.042	3.43	0.19	0.32	59.62	
E-3 (2)	393.0	395.0	385.0	383.0	10.0	1200	8	0.0017		0.05	0.04	0.000	0.000	3.43	0.19	0.32	59.62	
E-1 (1)	395.0	317.7	383.0	307.0	11.3	1000	8	0.076	209	0.05	0.04	0.230	0.209	2.96	1.02	2.15	47.33	
E-1 (2)	317.7	240.3	307.0	231.0	10.0	1000	8	0.076		0.28	0.25	0.000	0.000	2.96	1.02	2.15	47.33	
E-1 (3)	240.3	163.0	231.0	155.0	8.7	1000	8	0.076		0.28	0.25	0.000	0.000	2.96	1.02	2.15	47.33	
E-6	391.0	355.0	383.0	347.0	8.0	1100	8	0.0327	28	0.00	0.00	0.031	0.028	3.53	0.13	1.41	9.18	
Kato Lane/Mountainview Road Trunk																		
E-9 (1)	522.0	500.0	514.0	492.0	8.0	1000	8	0.022	140	0.00	0.00	0.154	0.140	3.11	0.59	1.16	50.91	
E-9 (2)	500.0	340.0	492.0	332.0	8.0	1250	8	0.128		0.15	0.14	0.000	0.000	3.11	0.59	2.79	21.10	
E-9 (3)	340.0	230.0	332.0	222.0	8.0	1250	8	0.088		0.15	0.14	0.000	0.000	3.11	0.59	2.32	25.45	
E-7	235.0	207.0	227.0	199.0	8.0	800	8	0.035	63	0.15	0.14	0.069	0.063	3.02	0.84	1.46	57.18	
E-4 (1)	305.0	207.0	297.0	199.0	8.0	1200	8	0.0817	30	0.00	0.00	0.033	0.030	3.51	0.14	2.23	6.20	1/3 of E-4 basin
E-4 (2)	207.0	195.0	199.0	172.5	15.3	1100	8	0.0241	60	0.22	0.20	0.066	0.060	2.95	1.06	1.21	87.82	2/3 of E-4 basin
E-4 (3)	195.0	150.0	172.5	142.0	15.3	1000	8	0.0305		0.29	0.26	0.000	0.000	2.95	1.06	1.36	78.05	
E-4 (4)	150.0	158.0	142.0	139.0	13.5	1100	12	0.0027		0.29	0.26	0.000	0.000	2.95	1.06	1.20	88.53	
State Highway 9 Trunk, cont'd																		
E-1 (4)	163.0	117.0	146.0	109.0	12.5	2300	15	0.0161		0.89	0.56	0.000	0.000	2.75	2.43	5.30	45.97	
E-1 (5)	117.0	100.0	109.0	85.0	11.5	3600	15	0.0067		0.89	0.56	0.000	0.000	2.75	2.43	3.41	71.42	
N10-C (1)	100.0	43.0	85.0	22.0	18.0	4000	15	0.0158	37	1.19	1.08	0.041	0.037	2.57	4.11	5.24	78.43	
E-2	382.0	379.0	374.0	371.0	8.0	1500	8	0.002	35	0.00	0.00	0.039	0.035	3.47	0.16	0.35	45.83	
N10-H	379.0	360.0	371.0	352.0	8.0	1750	8	0.0109	55	0.04	0.04	0.061	0.055	3.23	0.39	0.81	47.86	
N10-E	360.0	300.0	352.0	292.0	8.0	1000	8	0.06	44	0.099	0.090	0.048	0.044	3.12	0.57	1.91	29.58	
N10-C (2)	300.0	100.0	292.0	92.0	8.0	1400	8	0.1429	37	0.14	0.04	0.041	0.037	3.29	0.41	2.95	14.05	
E-5 (1)	394.0	391.3	386.0	383.3	8.0	1000	8	0.0027	57	0.00	0.00	0.063	0.057	3.35	0.25	0.41	62.45	
E-5 (2)	391.3	388.6	383.3	380.6	8.0	1000	8	0.0027		0.06	0.06	0.000	0.000	3.35	0.25	0.41	62.45	
E-5 (3)	388.6	385.9	380.6	377.9	8.0	1000	8	0.0027		0.06	0.06	0.000	0.000	3.35	0.25	0.41	62.45	

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
E-10		2.7							78	0.00	0.00	0.086	0.078					no individual collector pipe

APPENDIX B

Example of TABULA Cost Output and Additional Information

Cost Calculations for Pipe: **8" - 8' depth**

Project year: 2003

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2003
Length: 1000 ft
Conduit Type: Gravity Sewer
Depth of Cover: 8 ft
Trench Backfill Type: Native
Manhole Spacing: Average (500 ft)
Existing Utilities: Average
Dewatering: Minimal
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in.

Geometry

Outer Diameter	0.875 ft
Trench Width	3.64 ft
Excavation Depth	9.88 ft
Complete Surface Rest. Width	5.64 ft

Unit Costs (Basis 1999)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	1,330	CY	10.00	13,300
Backfill	943	CY	5.00	4,720
Complete Pavement Restoration	626	SY	50.00	31,300
Trench Safety	19,750	SF	0.50	9,880
Spoil Load and Haul	387	CY	10.00	3,870
Pipe Unit Material Cost	1,000	lf	10.00	10,000
Pipe Installation	1,000	lf	10.00	10,000
Place Pipe Zone Fill	365	CY	25.00	9,130
Manholes	2	MH	3,000.00	6,000
Existing Utilities	1,000	lf	20.00	20,000
Dewatering	1,000	lf	20.00	20,000
Traffic Control	1,000	lf	5.00	5,000

Year 1999 subtotal	143,000
Mobilization/Demobilization at 10%	1.10
1999 to 2003	1.13
Effective Multiplier	1.24
Subtotal	177,000

Total: $177,000

Cost Calculations for Pipe: **8" - 10' depth**

Project year: 2003

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2003
Length: 1000 ft
Conduit Type: Gravity Sewer
Depth of Cover: 10 ft
Trench Backfill Type: Native
Manhole Spacing: Average (500 ft)
Existing Utilities: Average
Dewatering: Minimal
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in.

Geometry

Outer Diameter	0.875 ft
Trench Width	3.64 ft
Excavation Depth	11.9 ft
Complete Surface Rest. Width	5.64 ft

Unit Costs (Basis 1999)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	1,600	CY	10.00	16,000
Backfill	1,213	CY	5.00	6,060
Complete Pavement Restoration	626	SY	50.00	31,300
Trench Safety	23,750	SF	0.50	11,900
Spoil Load and Haul	387	CY	10.00	3,870
Pipe Unit Material Cost	1,000	lf	10.00	10,000
Pipe Installation	1,000	lf	10.00	10,000
Place Pipe Zone Fill	365	CY	25.00	9,130
Manholes	2	MH	3,000.00	6,000
Existing Utilities	1,000	lf	20.00	20,000
Dewatering	1,000	lf	20.00	20,000
Traffic Control	1,000	lf	5.00	5,000

Year 1999 subtotal	149,000
Mobilization/Demobilization at 10%	1.10
1999 to 2003	1.13
Effective Multiplier	1.24
Subtotal	185,000

Total: $185,000

Cost Calculations for Pipe: **8" - 12' depth**

Project year: 2003

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2003
Length: 1000 ft
Conduit Type: Gravity Sewer
Depth of Cover: 12 ft
Trench Backfill Type: Native
Manhole Spacing: Average (500 ft)
Existing Utilities: Average
Dewatering: Minimal
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in.

Geometry

Outer Diameter	0.875 ft
Trench Width	3.64 ft
Excavation Depth	13.9 ft
Complete Surface Rest. Width	5.64 ft

Unit Costs (Basis 1999)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	1,869	CY	10.00	18,700
Backfill	1,482	CY	5.00	7,410
Complete Pavement Restoration	626	SY	50.00	31,300
Trench Safety	27,750	SF	0.50	13,900
Spoil Load and Haul	387	CY	10.00	3,870
Pipe Unit Material Cost	1,000	lf	10.00	10,000
Pipe Installation	1,000	lf	10.00	10,000
Place Pipe Zone Fill	365	CY	25.00	9,130
Manholes	2	MH	3,000.00	6,000
Existing Utilities	1,000	lf	20.00	20,000
Dewatering	1,000	lf	20.00	20,000
Traffic Control	1,000	lf	5.00	5,000

	Year 1999 subtotal	155,000
Mobilization/Demobilization at 10%		1.10
	1999 to 2003	1.13
	Effective Multiplier	1.24
	Subtotal	192,000

Total: $192,000

Cost Calculations for Pipe: **8" - 15' depth**

Project year: 2003

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2003
Length: 1000 ft
Conduit Type: Gravity Sewer
Depth of Cover: 15 ft
Trench Backfill Type: Native
Manhole Spacing: Average (500 ft)
Existing Utilities: Average
Dewatering: Minimal
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in.

Geometry

Outer Diameter	0.875 ft
Trench Width	3.64 ft
Excavation Depth	16.9 ft
Complete Surface Rest. Width	5.64 ft

Unit Costs (Basis 1999)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	2,273	CY	10.00	22,700
Backfill	1,886	CY	5.00	9,430
Complete Pavement Restoration	626	SY	50.00	31,300
Trench Safety	33,750	SF	0.50	16,900
Spoil Load and Haul	387	CY	10.00	3,870
Pipe Unit Material Cost	1,000	lf	10.00	10,000
Pipe Installation	1,000	lf	10.00	10,000
Place Pipe Zone Fill	365	CY	25.00	9,130
Manholes	2	MH	3,750.00	7,500
Existing Utilities	1,000	lf	20.00	20,000
Dewatering	1,000	lf	20.00	20,000
Traffic Control	1,000	lf	5.00	5,000

Year 1999 subtotal	166,000
Mobilization/Demobilization at 10%	1.10
1999 to 2003	1.13
Effective Multiplier	1.24
Subtotal	205,000

Total: $205,000

Appendix B – Example of TABULA Cost Output and Additional Information

The preceding examples are for 8-inch sewer pipe at different depths.

For additional information or to download the TABULA software, go to http://www.bugbytes.com/tabula/.

APPENDIX C

UGA Cost Estimates

Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost [1]	Taxes (8.8%)	Contingency [2] (40%)	Engineering and Administration Costs [3] (25%)	Project Cost [4]
North UGA										
N15-A (1)	9.5	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	**$ 360,000**
N15-A (2)	9.5	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	**$ 360,000**
N15-A (3)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N15-A (4)	8.0	1300	8	LF	$ 177	$ 230,100	$ 20,249	$ 100,140	$ 87,622	**$ 440,000**
N15-C (1)	9.7	700	10	LF	$ 195	$ 136,500	$ 12,012	$ 59,405	$ 51,979	**$ 260,000**
N15-C (2)	12.9	1000	10	LF	$ 203	$ 203,000	$ 17,864	$ 88,346	$ 77,302	**$ 390,000**
N15-C (3)	15.1	600	10	LF	$ 216	$ 129,600	$ 11,405	$ 56,402	$ 49,352	**$ 250,000**
N15-B (1)	9.1	1100	8	LF	$ 185	$ 203,500	$ 17,908	$ 88,563	$ 77,493	**$ 390,000**
N15-B (2)	12.9	1100	8	LF	$ 192	$ 211,200	$ 18,586	$ 91,914	$ 80,425	**$ 410,000**
N15-B (3)	11.8	1100	8	LF	$ 192	$ 211,200	$ 18,586	$ 91,914	$ 80,425	**$ 410,000**
N15-B (4)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N15-B (5)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N15-B (6)	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
N15-D (1)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
N15-D (2)	8.0	1250	10	LF	$ 187	$ 233,750	$ 20,570	$ 101,728	$ 89,012	**$ 450,000**
N15-D (3)	14.5	1000	10	LF	$ 216	$ 216,000	$ 19,008	$ 94,003	$ 82,253	**$ 420,000**
N15-D (4)	14.5	800	12	LF	$ 238	$ 190,400	$ 16,755	$ 82,862	$ 72,504	**$ 370,000**
N15-D (5)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N16 (1)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N16 (2)	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
N9-F (1)	8.0	600	8	LF	$ 177	$ 106,200	$ 9,346	$ 46,218	$ 40,441	**$ 210,000**
N9-F (2)	8.0	1100	8	LF	$ 177	$ 194,700	$ 17,134	$ 84,733	$ 74,142	**$ 380,000**
PS 1/N15-B & 3,400 lf FM		390		GPM	$ 2,100	$ 819,000	$ 72,072	$ 356,429	$ 311,875	**$ 1,560,000**
PS 2/N16 & 1,600 lf FM		150		GPM	$ 4,000	$ 600,000	$ 52,800	$ 261,120	$ 228,480	**$ 1,150,000**
PS 3/N15-D & 600 lf FM		875		GPM	$ 700	$ 612,500	$ 53,900	$ 266,560	$ 233,240	**$ 1,170,000**
						$ 6,030,000				**$ 11,580,000**

Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost [(1)]	Taxes (8.8%)	Contingency [(2)] (40%)	Engineering and Administration Costs [(3)] (25%)	Project Cost [(4)]
South UGA										
S5-B (1)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
S5-B (2)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
S5-B (3)	8.0	990	10	LF	$ 187	$ 185,130	$ 16,291	$ 80,569	$ 70,498	**$ 360,000**
S5-B (4)	8.0	820	10	LF	$ 187	$ 153,340	$ 13,494	$ 66,734	$ 58,392	**$ 300,000**
S4-A2 (1)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
S4-A2 (2)	8.0	600	10	LF	$ 187	$ 112,200	$ 9,874	$ 48,829	$ 42,726	**$ 220,000**
S4-A2 (3)	7.5	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
S4-A2 (4)	8.0	950	8	LF	$ 177	$ 168,150	$ 14,797	$ 73,179	$ 64,032	**$ 330,000**
S4-A2 (5)	8.0	550	8	LF	$ 177	$ 97,350	$ 8,567	$ 42,367	$ 37,071	**$ 190,000**
S4-A2 (6)	8.0	600	8	LF	$ 177	$ 106,200	$ 9,346	$ 46,218	$ 40,441	**$ 210,000**
S3-A2 (1)	8.0	1100	10	LF	$ 187	$ 205,700	$ 18,102	$ 89,521	$ 78,331	**$ 400,000**
S4-A1 (1)	10.5	1000	10	LF	$ 203	$ 203,000	$ 17,864	$ 88,346	$ 77,302	**$ 390,000**
S4-A1 (2)	10.5	1000	10	LF	$ 203	$ 203,000	$ 17,864	$ 88,346	$ 77,302	**$ 390,000**
S4-A1 (3)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
S3-A2 (2)	8.0	500	8	LF	$ 177	$ 88,500	$ 7,788	$ 38,515	$ 33,701	**$ 170,000**
S3-A2 (3)	8.0	650	8	LF	$ 177	$ 115,050	$ 10,124	$ 50,070	$ 43,811	**$ 220,000**
S3-A1 (1)	8.0	1200	10	LF	$ 187	$ 224,400	$ 19,747	$ 97,659	$ 85,452	**$ 430,000**
S3-A1 (2)	13.5	1000	10	LF	$ 216	$ 216,000	$ 19,008	$ 94,003	$ 82,253	**$ 420,000**
S3-A1 (3)	8.0	1700	10	LF	$ 187	$ 317,900	$ 27,975	$ 138,350	$ 121,056	**$ 610,000**
S3-B2 (1)	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
S3-B2 (2)	8.5	900	8	LF	$ 185	$ 166,500	$ 14,652	$ 72,461	$ 63,403	**$ 320,000**
S4-B (1)	8.0	1400	8	LF	$ 177	$ 247,800	$ 21,806	$ 107,843	$ 94,362	**$ 480,000**
S4-B (2)	8.0	1800	8	LF	$ 177	$ 318,600	$ 28,037	$ 138,655	$ 121,323	**$ 610,000**
S4-B (3)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
S4-B (4)	8.0	1200	8	LF	$ 177	$ 212,400	$ 18,691	$ 92,436	$ 80,882	**$ 410,000**
S4-B (5)	8.0	1100	8	LF	$ 177	$ 194,700	$ 17,134	$ 84,733	$ 74,142	**$ 380,000**
S4-B (6)	8.0	1200	8	LF	$ 177	$ 212,400	$ 18,691	$ 92,436	$ 80,882	**$ 410,000**

City of Mount Vernon

Urban Growth Area Sewer Service Study

October 2003

Prepared by:

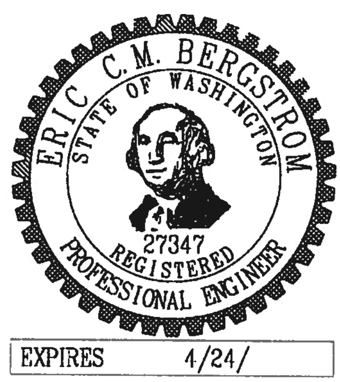
HDR

CERTIFICATION PAGE

FOR

City of Mount Vernon
Urban Growth Area Sewer Service Study
Project No. 000 000 000 005237 002

The engineering material and data contained in this Report were prepared under the supervision and direction of the undersigned, whole seal as registered professional engineer is affixed below.



Eric C.M. Bergstrom
Supervising Engineer

City of Mount Vernon

Urban Growth Area Sewer Service Study
Technical Memorandum

Date: October 15, 2003

To: Walt Enquist, Fred Buckenmeyer

From: Eric Bergstrom

Subject: Urban Growth Area Sewer Service Study

INTRODUCTION AND OBJECTIVE

The City is required to establish a plan to provide sewer service to properties within the Urban Growth Area (UGA). This study identifies at a planning level the facilities that would be required to provide sewer service to four major areas within the UGA. These UGA areas are situated to the north, south, east and west of City limits and as illustrated in Figure 1. The areas vary in size from 175 to 1,400 acres.

DRAINAGE BASIN DELINEATION

Each of the UGA service areas shown in Figure 1 was divided into a number of drainage basins. The drainage basins were delineated based on contour and mapping data provided by City of Mt. Vernon geographical information system (GIS) as well as some field reconnaissance. Figures 2, 3, 4, and 5 show the approximate drainage basin boundaries within north, south, east and west UGAs, respectively. Each drainage basin is designated an ID such as E-12, which serves to clarify new sewer locations. The first letter of basin ID refers to a specific UGA, i.e. S for south etc. The basin ID's follow a format set forth in the *City of Mount Vernon Comprehensive Sewer Plan Update*, February 2003.

DESIGN FLOWS

Design flows from each of the drainage basins were estimated based on an assumed population density of 2.5 persons per house and a house density of 4 houses per acre. Average daily flow per capita was assumed to be 100 gallons. Consistent with Comprehensive Sewer Plan, a peaking factor was applied to predict peak daily sanitary flow. The peaking factor for the sanitary flows at any point in the system is based on the following equation:

$$PF = -0.6 \log Q + 2.6$$

where,
PF peaking factor
Q average sanitary flow in million gallons per day (mgd)

Inflow and infiltration (I&I), is independent of sanitary flow and is assumed to be 1,100 gallons per acre per day (gpda). The sanitary sewer peaking factor is not applied to the allowance for I&I.

Table 1 summarizes the drainage basins and calculated average and peak flow for each basin.

Table 1
Drainage Basins Flows

Drainage Basin ID[1]	Basin Area (acres)	Estimated Population[2]	Average Sanitary Flow (gpm)	Peaking Factor	Inflow and Infiltration (gpm)	Peak Design Flow (gpm)
North UGA						
N11-T	26	260	18	3.6	20	84
N15-A	101	1007	70	3.2	77	300
N15-B	132	1320	92	3.1	101	388
N15-C	64	640	44	3.3	49	196
N15-D	59	590	41	3.3	45	182
N16	47	469	33	3.4	36	146
N9-F	25	249	17	3.6	19	80
South UGA						
S1-D	141	1407	98	3.1	107	411
S3-A1	69	689	48	3.3	53	211
S3-A2	167	1672	116	3.1	128	484
S3-B2	84	835	58	3.2	64	252
S3-C	139	1389	96	3.1	106	407
S4-A1	66	661	46	3.3	50	202
S4-A2	99	988	69	3.2	75	295
S4-B	124	1236	86	3.1	94	364
S4-C	274	2737	190	2.9	209	767
S5-B	185	1854	129	3.0	142	533
East UGA						
E-1	209	2088	145	3.0	160	596
E-10	172	1718	119	3.1	131	496
E-11	90	896	62	3.2	68	269
E-12	104	1044	73	3.2	80	311
E-13	129	1286	89	3.1	98	378
E-14	86	861	60	3.2	66	259
E-15	40	400	28	3.4	31	126
E-2	35	347	24	3.5	27	110
E-3	42	416	29	3.4	32	131
E-4	90	903	63	3.2	69	271
E-5	57	575	40	3.3	44	177
E-6	28	285	20	3.5	22	92
E-7	63	633	44	3.3	48	194
E-8	178	1782	124	3.0	136	513
E-9	140	1400	97	3.1	107	410
N10-C	37	375	26	3.5	29	119
N10-E	44	439	30	3.4	34	138
N10-H	55	548	38	3.4	42	169
West UGA						
W1	178	1781	124	3.0	136	513
W2-A	98	977	68	3.2	75	292
W2-B	75	751	52	3.3	57	228

(1) See Figures 2 through 5 for drainage basin designations in each UGA.
(2) Saturated development.

SEWER LAYOUT AND SIZING

Figures 6, 7, 8, and 9 illustrate proposed sewer locations within the UGAs. Where possible, proposed sewers were located in existing right-of-ways. In several locations proposed sewers are shown outside of right-of-way boundaries where these routes provide a more cost effective alignment. Some sewers are shown outside the boundary of the East UGA in order maximize the natural drainage of the area. For this study it was assumed that the sewers outside the UGA boundaries are strictly for conveyance and there are no service connections along the alignment. The piping shown on the figures only includes trunk sewers and interceptors. Small collector sewers that would serve individual properties generally have not been identified.

After proposed alignment of interceptors and major trunk lines was established, grade elevations at approximately 1,000-foot intervals were established based on City's contour mapping. For the purposes of this study it was assumed that the minimum pipe depth to the invert would be 8 feet below grade. In some cases, in order to provide greater slope to improve the flow, greater depths of pipe were assumed. The hydraulic analysis of proposed sewers was completed based on fully developed UGAs for each individual drainage basin. In the model the flows from the drainage basin or portions of the drainage basin are routed into the upstream end of pipe segments.

PROPOSED IMPROVEMENTS

Based on the UGA evaluation proposed sewer improvements were identified. Table 2 summarizes the improvements proposed for each UGA. For detailed list of improvements refer to Appendix A. Figures 6 through 9 illustrate the location of the sewers, forcemains and pump stations and show sewer sizing within each basin.

Table 2
Summary of Proposed Improvements for all UGAs

UGA	Sewer (linear feet)	Pump Stations (ea)	Force Main (linear feet)
North	22,000	3	5,000
South	45,000	2	2,000
East	53,000	1	5,000
West	12,000	1	800

Improvement S3-A2 (1) extends sewer service to intercept flows from basin S4-A2. Initially, sewer service in this basin can be served through SE-A2 (1). Ultimately, as the basin development proceeds, sewer flows will need to be routed down the proposed interceptor on the south side of Maddox Creek and includes sewer segments S4-A1 (1), (2), and (3) so that the sewers on South 19th Street do not become overloaded. The City should monitor flows in the sewer to determine when sewer segments S4-A1 (1), (2), and (3) are required.

Pump station S4-A2 PS1 has been identified at the end of Crosby Drive because the grade to the west may be excessive for a gravity sewer. It may be feasible to extend a gravity sewer to the west and connect to proposed sewer segments S4-A1 (1), (2), and (3) and the City should evaluate the potential for extending gravity service to Crosby Drive as development proceeds.

COST ESTIMATES

Construction Costs

Construction costs for the facilities required to provide sewer service to the Urban Growth Areas were based on cost established in the Comprehensive Sewer Plan Update as well as King County's cost estimating software TABULA. TABULA is a free software developed to estimate costs for sewer projects in the King County area. Use of TABULA allows for cost analysis that is always consistent. The costs can be adjusted to present day values using the ENR cost indexes and/or escalated to future years by using an annual projected inflation multiplier. The inflation multiplier can be defined in the program as any chosen percentage, however, 3 percent is commonly used. An effective 1.13 multiplier was used in the analysis to account for increase in construction cost from 1999 to the end of 2003.

For an example of TABULA's assumptions and item cost breakdown see Appendix B. Note that all assumptions can be defined within the program to better suit a specific project. Table 3 lists costs of pipe, at varying depths, based on TABULA. See Appendix C, column 7, for construction costs for major items associated with each UGA. Costs listed in Table 5 are also construction costs.

The construction costs include excavation and native backfill based on depth and pipe size, standard trench safety, backfill and pipe zone fill, manholes at average spacing of 500 feet, protecting/relocating average complexity utilities, minimal dewatering, traffic control for light traffic conditions, complete pavement restoration for width of trench, and mobilization/demobilization (10 percent of total). These particular costs do not include land acquisition or easements as these elements are difficult to define at the planning stages of design.

Table 3
Construction Cost Estimate Summary Table

Sewer Depth (ft)	Construction Cost per Liner Foot of Sewer								
	8-inch	10-inch	12-inch	15-inch	18-inch	24-inch	27-inch	30-inch	36-inch
8	$177	$187	$207	$237	$258	$316	$341	$385	$452
10	$185	$195	$215	$246	$267	$327	$351	$396	$465
12	$192	$203	$223	$255	$276	$337	$362	$408	$477
15	$205	$216	$238	$270	$292	$355	$381	$429	$500

Notes:
1) Costs are for year 2003.
2) Construction costs include mobilization, excavation, backfill and pipe zone fill, pavement restoration, trench safety, pipe material and installation, manholes, protecting/relocating existing utilities, dewatering, and traffic control.

The costs listed in Table 3 do not include taxes, construction contingency, or engineering and administration costs, which are briefly described below.

Contingency

The construction contingency is an allowance for additional costs not identified in the planning phase. Generally, these costs are not identified because they are unknown at the planning stages of the project. The contingency costs may include complex utility crossings, unique soil conditions, traffic control for heavy traffic conditions, land acquisition, easements, and other unidentified costs. See Appendix C, column 9, for contingency costs on construction costs for major items associated with each UGA.

Engineering and Administration

Engineering and administration costs include engineering and design, permitting process and fees, and City construction management costs. See Appendix C, column 10, for engineering and administration costs on construction costs for major items associated with each UGA.

Project Cost

Project costs include construction costs with an escalation for the following items:

- Sales Tax – 8.8 percent
- Contingency – 40 percent
- Engineering and Administration – 25 percent.

The major construction items and respective costs associated with each UGA are presented in Appendix C. See column 11 of Appendix C, for project costs on construction costs for major items associated with each UGA. Note that Figures 6 through 9 identify individual pipe segments, which are listed in Appendix C. The approximate construction and project costs estimated for each of the UGAs are summarized in Table 4. They include gravity and forcemain piping costs as well as pump station costs.

Table 4
Approximate UGA Construction and Project Costs

Urban Growth Area	Construction Cost (1) (2003)	Project Cost (2) (2003)
North	$6,030,000	$11,580,000
South	$10,510,000	$20,230,000
East	$15,720,000	$30,110,000
East (+ 1,200 ac) (3)	$8,540,000	$16,270,000
East (+ 5,600 ac) (3)	$12,840,000	$24,450,000
West	$2,820,000	$5,440,000

Notes:
(1) Construction costs include mobilization, excavation, backfill and pipe zone fill, pavement restoration, trench safety, pipe material and installation, manholes, protecting & relocating existing utilities, dewatering, and traffic control.
(2) Project costs include sales tax, contingency (unidentified item, such as complex utilities, soil conditions, traffic control, land acquisition, easements, etc.), and engineering and administration costs (engineering and design, permitting, management, etc.).
(3) The expended East service area is described in the proceeding section. Costs for additional 1,200 and 5,600 acres assumes PS1-E, E-8 (4), E-1(4), E-1(5), and N10-C(1) improvements only.

EXPANDED EAST SERVICE AREA

There is potential that sewer service may be provided east of the UGA. The impacts of this service on the sewer sizing within the City was evaluated for different size drainage areas to the east. Table 5 summarizes the impact on the existing sewers within the City along with the sizing of new facilities identified within this study.

Table 5
Expanded Service Area Impacts on Sewer Sizing

Improvement	Sewer Length (linear feet)	Assumed Slope (ft/ft)	Urban Growth Area Service: Design Flows (mgd)	Urban Growth Area Service: Pipe Size (in)	Urban Growth Area Service: Construction Cost [4] (million dollars)	1,200 Additional Acres: Design Flows (mgd)	1,200 Additional Acres: Pipe Size (in)	1,200 Additional Acres: Construction Cost (million dollars)	5,600 Additional Acres: Design Flows (mgd)	5,600 Additional Acres: Pipe Size (in)	5,600 Additional Acres: Construction Cost (million dollars)
PS1 - E	--	--	2.5	--	$2.74 [1]	6.6	--	$4.50 [1]	20.1	--	$7.01 [1]
Forcemain	4,300	--	2.5	12	--	6.6	24[5]	--	20.1	36[5]	--
E-8 (4)	2,000	0.008	2.5	15	$0.51	6.6	24	$0.67	20.1	36	$0.95
E-1 (4)	2,300	0.016	2.7	15	$0.59	7.3	24	$0.78	20.9	36	$1.10
E-1 (5)	3,600	0.007	2.7	15	$0.92	7.3	24	$1.21	20.9	36	$1.72
N10-C (1)	4,000	0.016	4.7	15	$1.08	8.3	24	$1.38	21.8	36	$2.06
East College Way Pump Station	--	--	9.6	--		13.3	--		24.4	--	
East College Way Pump Station Forcemain	6,300	--	9.6 [2]	24		13.3	30		24.4	36	
Kulshan Interceptor	6,860	0.006	18.9 [2]	24 & 30 [3]	--	22.5	30		35.1	36	

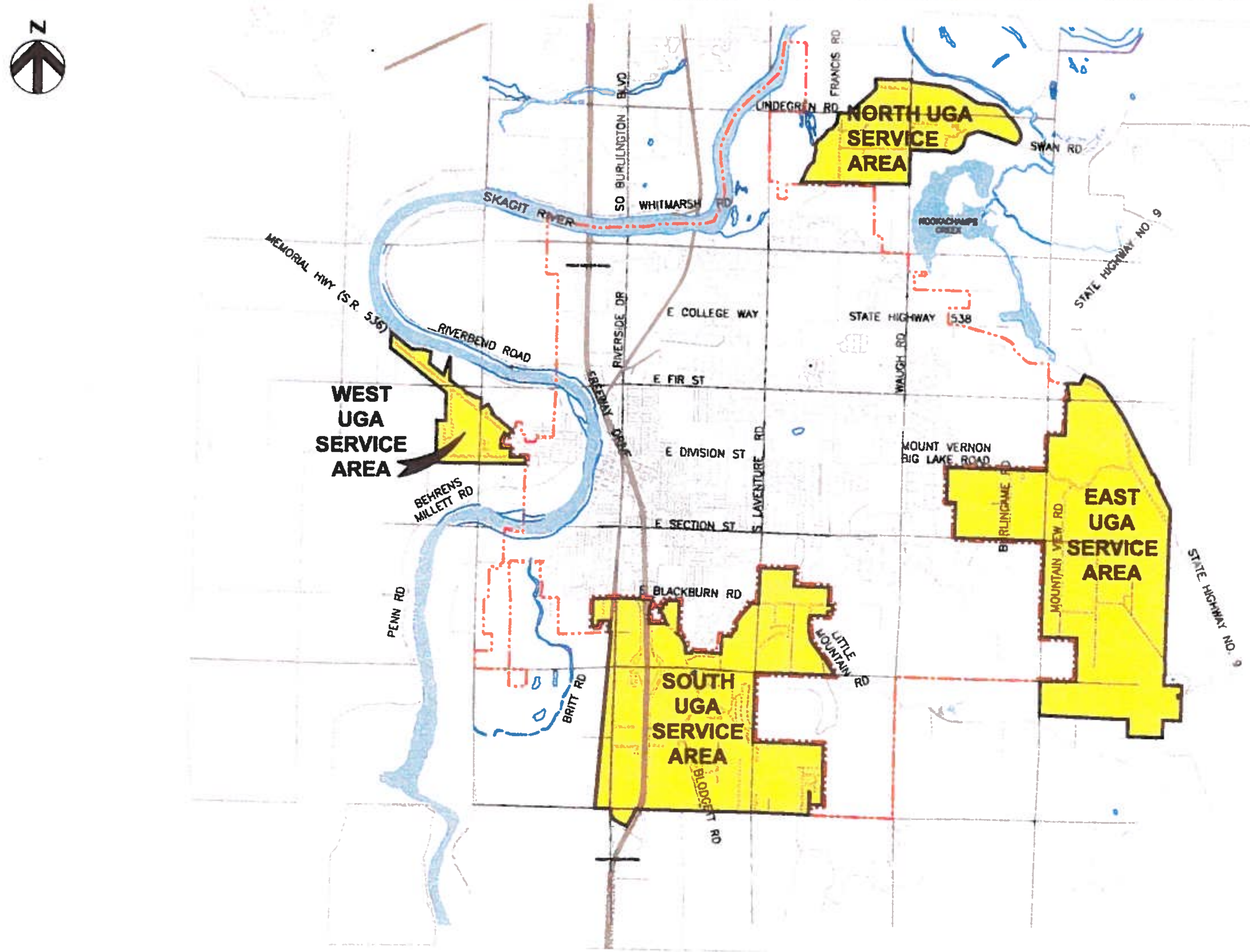
Notes:
(1) Cost Includes FM.
(2) Existing flows plus East UGA flows.
(3) Existing piping at capacity is approximately 18 mgd.
(4) Construction costs include mobilization, excavation, backfill and pipe zone fill, pavement restoration, trench safety, pipe material and installation, manholes, protecting/relocating existing utilities, dewatering, and traffic control.
(5) Multiple forcemains could be constructed with this effective pipe size diameter.

Note that costs listed Table 5 are construction costs only. See Appendix B for cost escalations to arrive at project costs.

LIST OF APPENDICES

APPENDIX A

Hydraulic Analysis Output of the City of Mount Vernon's Urban Growth Area Wastewater Collection System



4000 0 4000 8000

SCALE IN FEET

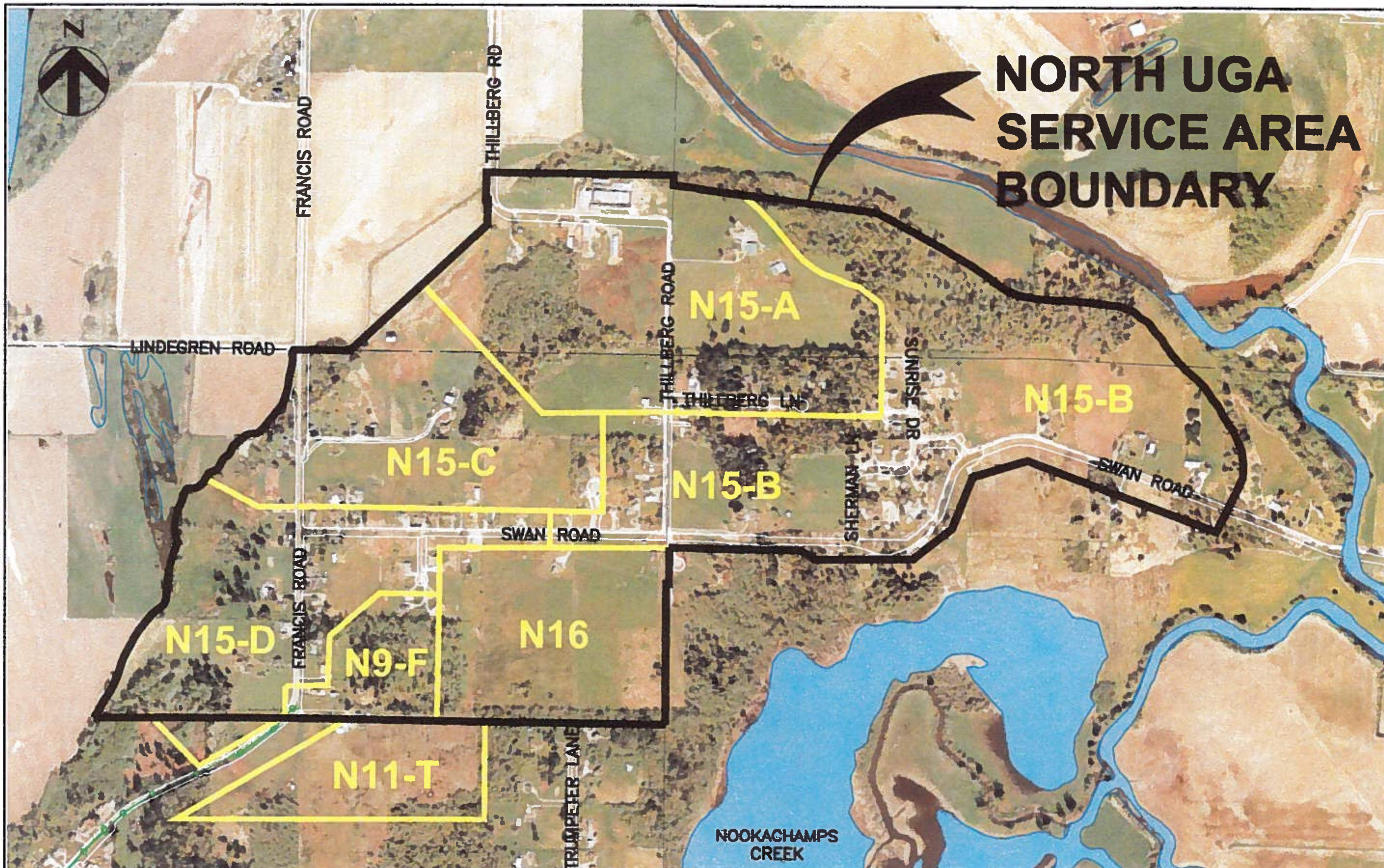
LEGEND

CITY LIMITS

URBAN GROWTH AREA (UGA) EXPANSION

HDR Engineering, Inc.

Project Title: CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA	Date: SEPTEMBER 2003
Sheet Title: URBAN GROWTH AREA (UGA) EXPANSION	Figure No. 1



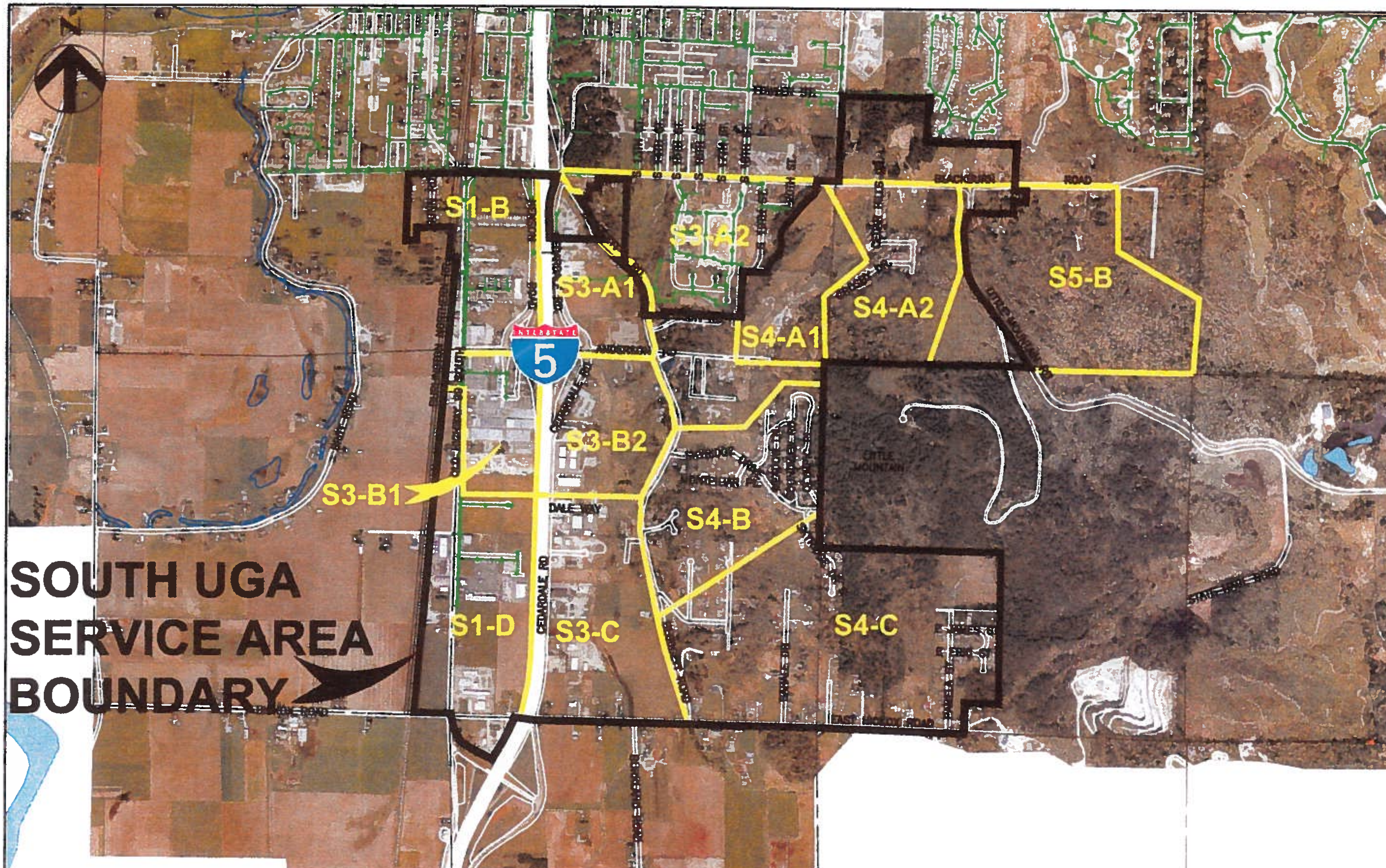
LEGEND

DRAINAGE BASIN BOUNDARY
EXISTING SEWER

HDR Engineering, Inc.

Project Title	CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA	Figure No. 2
Sheet Title	URBAN GROWTH AREA (UGA) NORTH SERVICE AREA DRAINAGE BASINS	Date AUGUST 2003
		Scale 1" = 1000'

MV-UGA-NORTH-FIG2A.dwg 06/20/03 BLUMENFELD



LEGEND

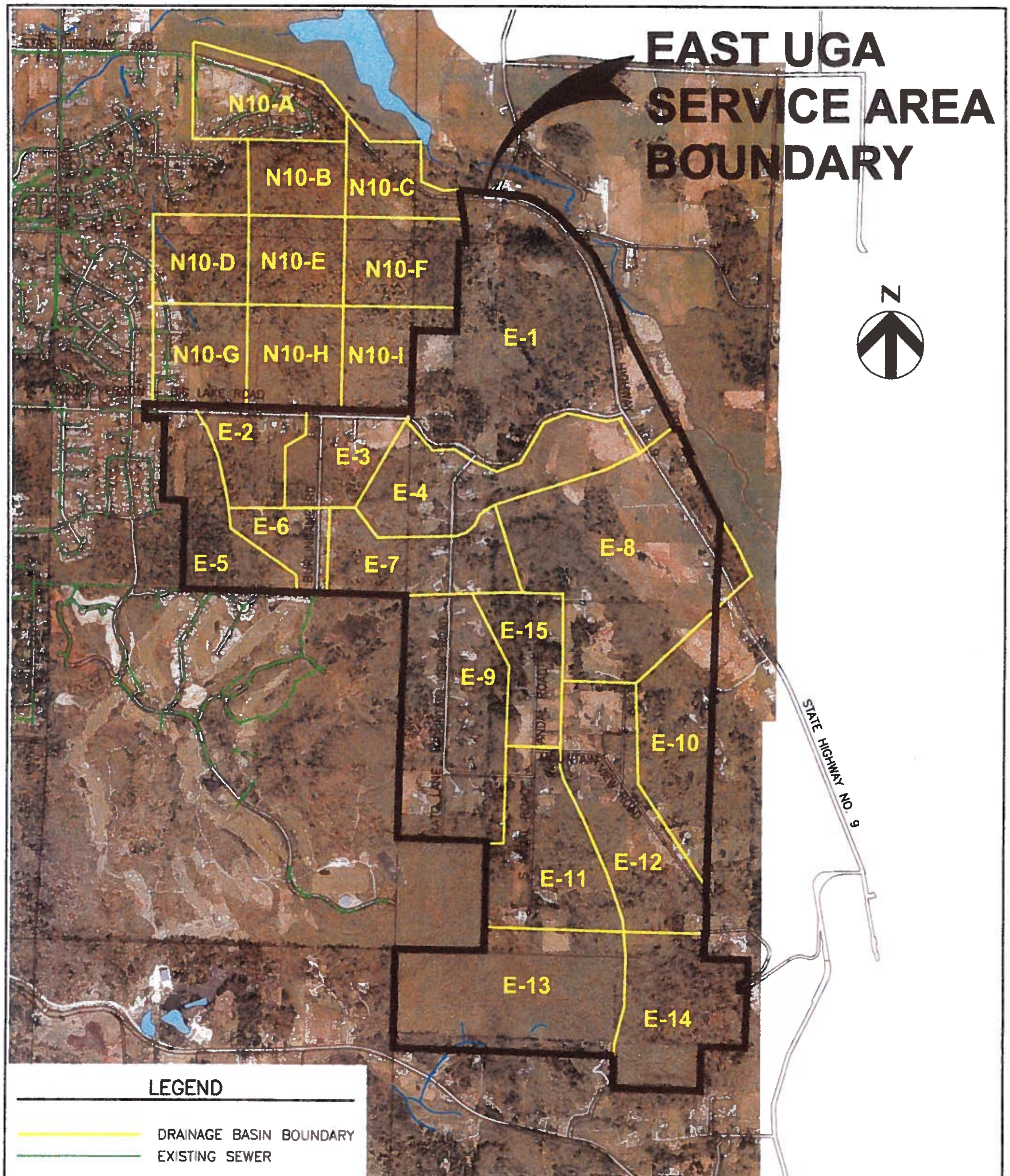
DRAINAGE BASIN BOUNDARY

EXISTING SEWER

HDR Engineering, Inc.

Project Title	CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA
Sheet Title	URBAN GROWTH AREA (UGA) SOUTH SERVICE AREA DRAINAGE BASINS
Figure No.	3
Date	SEPTEMBER 2003
Scale	1" = 2000'

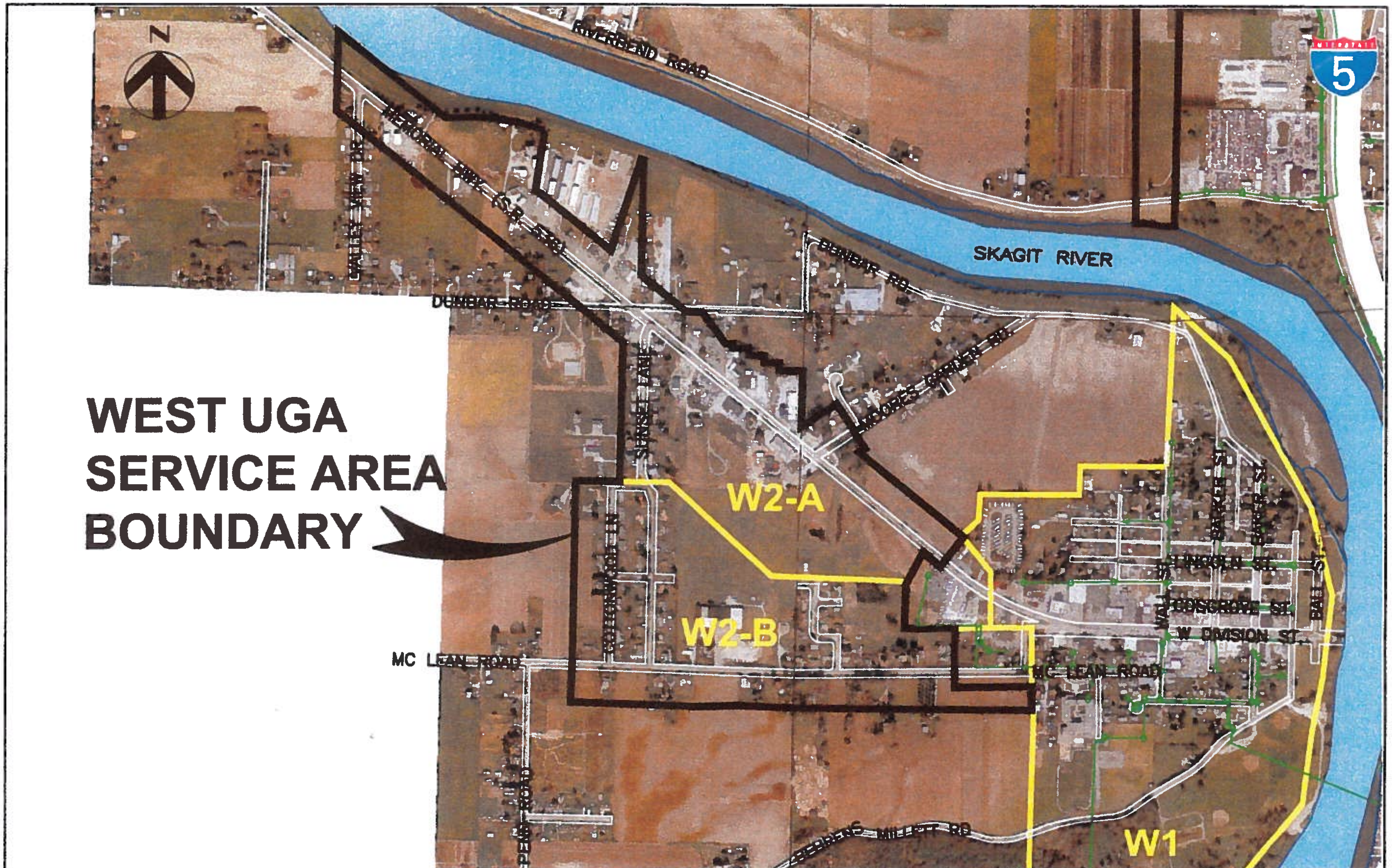
MV-UGA-SOUTH-FIG3.dwg 09/12/03 GUNDY



Project Title	CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA	Figure No 4
Sheet Title	URBAN GROWTH AREA (UGA) DRAINAGE BASINS EAST SERVICE AREA	Date AUGUST 2003
		Scale 1" = 2000'

HDR Engineering, Inc.

MV-UGA-EAST-FIG5A.dwg 08/20/03 BLUMENFELD



LEGEND

DRAINAGE BASIN BOUNDARY

EXISTING SEWER

HDR Engineering, Inc.

Project Title: CITY OF MOUNT VERNON
SEWER SERVICE STUDY FOR URBAN GROWTH AREA

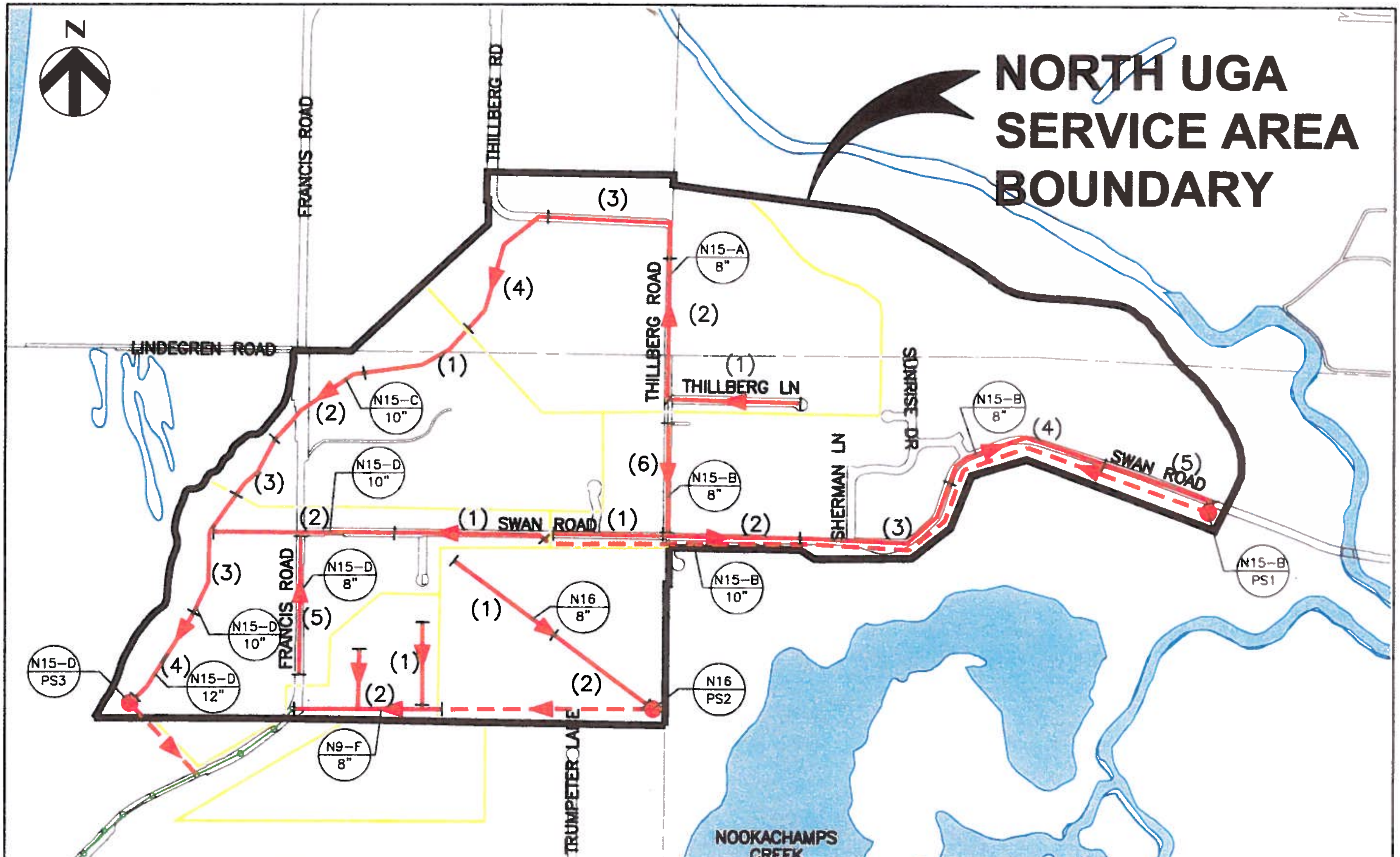
Sheet Title: URBAN GROWTH AREA (UGA)
WEST SERVICE AREA
DRAINAGE BASINS

Figure No. 5

Date: AUGUST 2003

Scale: 1" = 1000'

MV-UGA-WEST-FIG5.dwg 08/20/03 BLUMENFELD



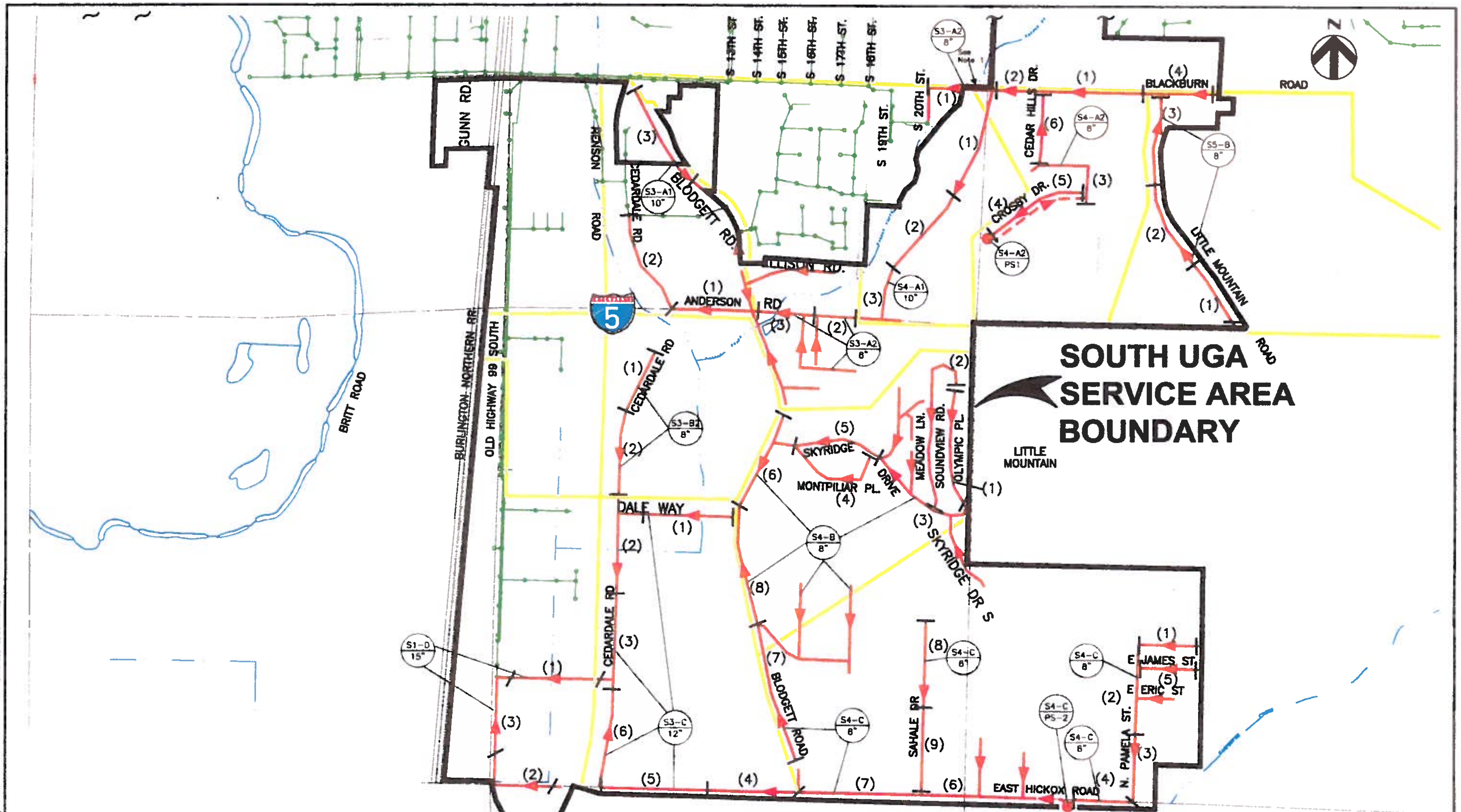
LEGEND

- DRAINAGE BASIN BOUNDARY
- EXISTING SEWER
- PROPOSED SEWER
- PROPOSED SEWER FORCEMAIN
- N15–D / 12" — DRAINAGE BASIN ID / PIPE SIZE B/W TICKS OR PUMP STA #
- PROPOSED PUMP STATION
- DIRECTION OF FLOW

HDR Engineering, Inc.

Project Title	CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA	Figure No.	6
Sheet Title	URBAN GROWTH AREA (UGA) NORTH SERVICE AREA PROPOSED SEWER PLAN	Date	AUGUST 2003
		Scale	1" = 1000'

MV-UGA-NORTH-FIG6.dwg 09/08/03 GUNDY



LEGEND

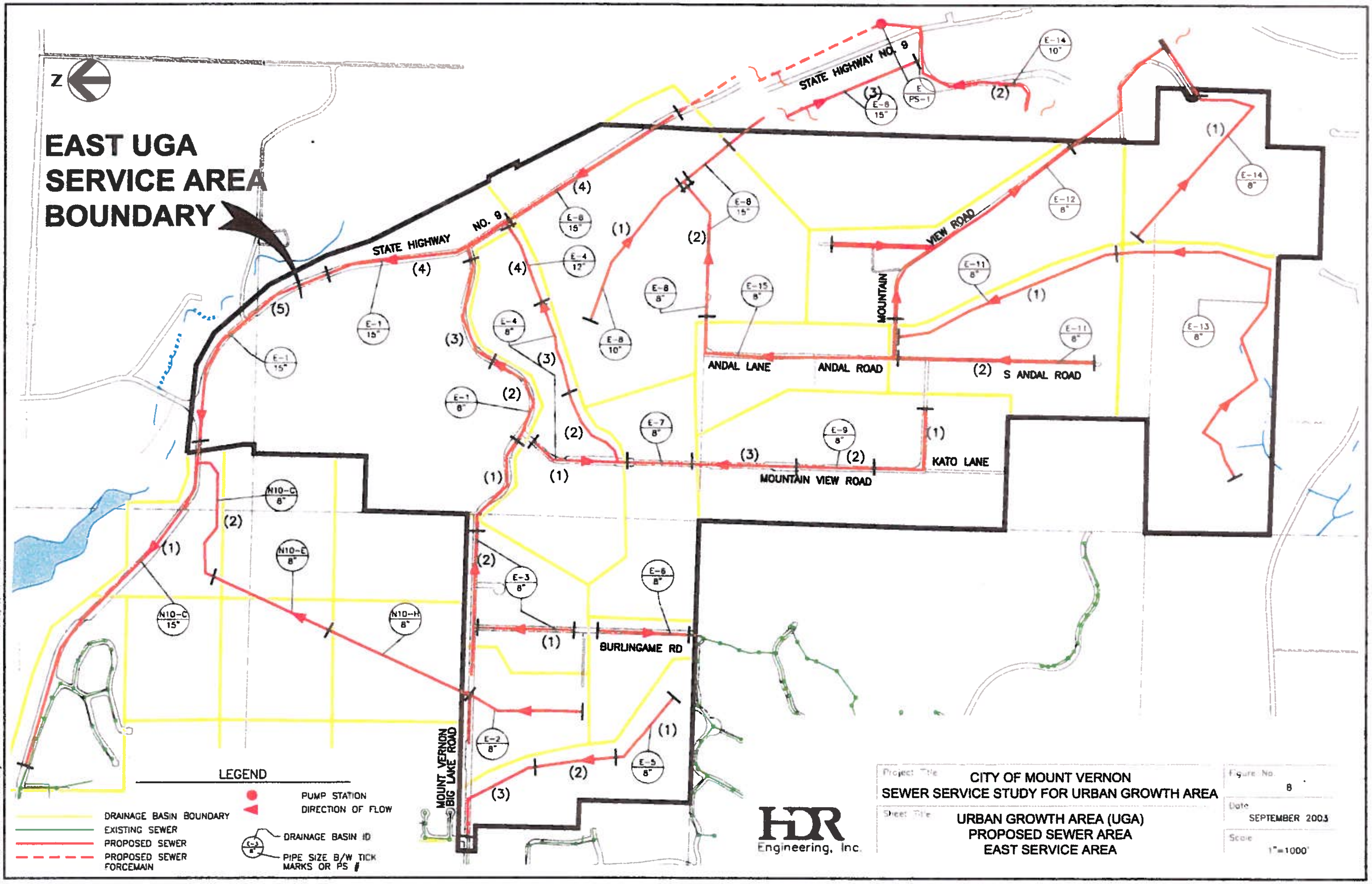
- DRAINAGE BASIN BOUNDARY
- EXISTING SEWER
- PROPOSED SEWER
- PROPOSED SEWER FORCEMAIN
- DRAINAGE BASIN ID / PIPE SIZE B/W TICKS OR PUMP STA #
- PUMP STATION
- DIRECTION OF FLOW

NOTES:

1. Sewer service to S4-A2 and S5-B may initially be provided via pipe segment S3-A2(1); however, as flows increase in the future, they will need to be diverted to S4-A1 (1), (2), and (3) to prevent overloading. The City should monitor flows to determine when pipe segments S4-A1 (1), (2), and (3) are required.

HDR Engineering, Inc.

Project Title	CITY OF MOUNT VERNON SEWER SERVICE STUDY FOR URBAN GROWTH AREA
Sheet Title	URBAN GROWTH AREA (UGA) SOUTH SERVICE AREA PROPOSED SEWER PLAN
Figure No.	7
Date	SEPTEMBER 2003
Scale	1" = 1000'


N
EAST UGA
SERVICE AREA
BOUNDARY
STATE HIGHWAY NO. 9
MOUNT VERNON BIG LAKE ROAD
BURLINGAME RD
MOUNTAIN VIEW ROAD
KATO LANE
ANDAL LANE
ANDAL ROAD
S ANDAL ROAD
MOUNTAIN
VIEW ROAD
LEGEND
DRAINAGE BASIN BOUNDARY
EXISTING SEWER
PROPOSED SEWER
PROPOSED SEWER FORCEMAIN
PUMP STATION
DIRECTION OF FLOW
DRAINAGE BASIN ID
PIPE SIZE B/W TICK MARKS OR PS #
HDR Engineering, Inc.
Project Title
CITY OF MOUNT VERNON
SEWER SERVICE STUDY FOR URBAN GROWTH AREA
Sheet Title
URBAN GROWTH AREA (UGA)
PROPOSED SEWER AREA
EAST SERVICE AREA
Figure No.
8
Date
SEPTEMBER 2003
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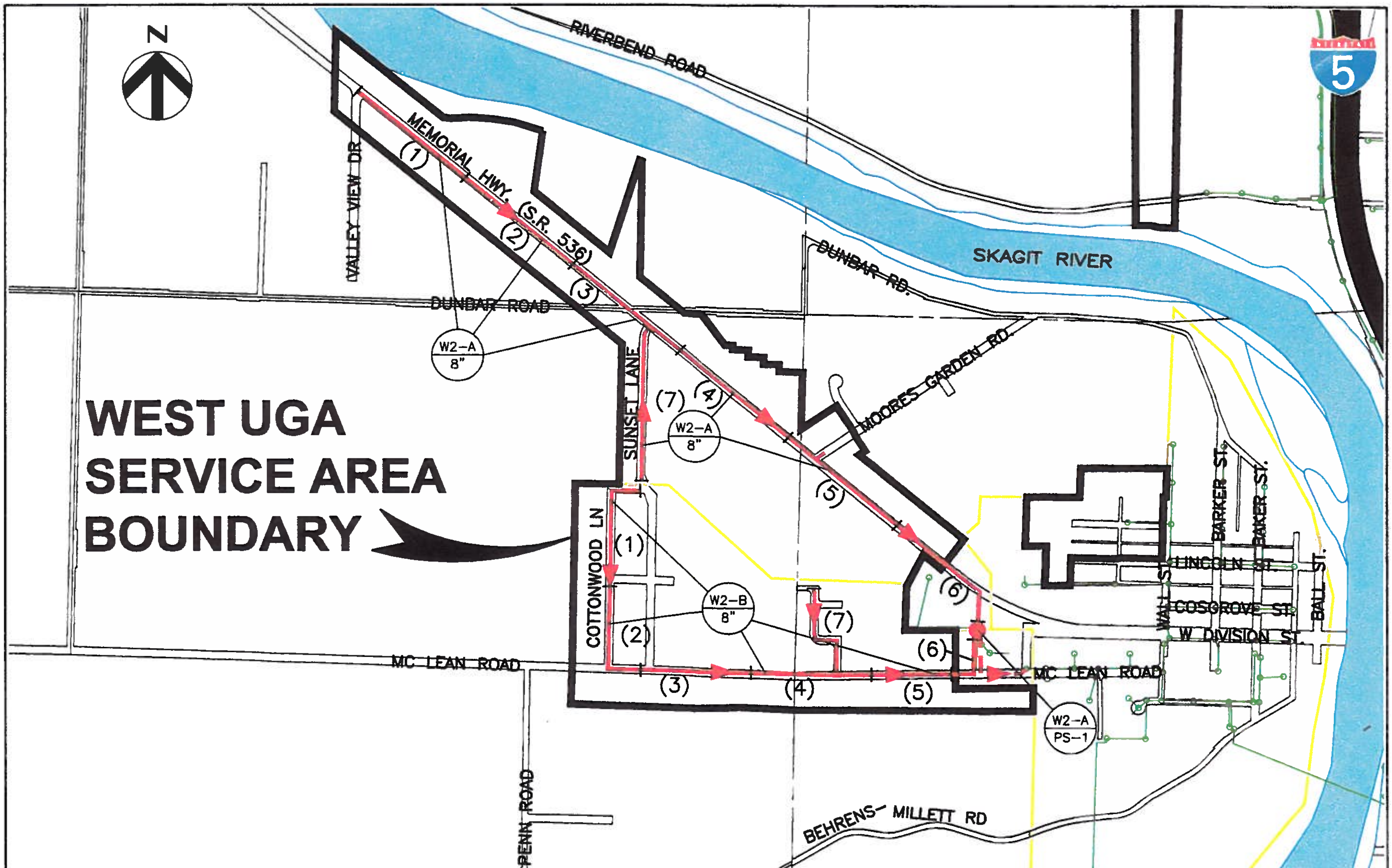

LEGEND

DRAINAGE BASIN BOUNDARY
EXISTING SEWER
PROPOSED SEWER
PROPOSED SEWER FORCEMAIN
DRAINAGE BASIN ID
PIPE SIZE B/W TICKS OR PUMP STA #
PROPOSED PUMP STATION
DIRECTION OF FLOW

HDR Engineering, Inc.

Project Title: CITY OF MOUNT VERNON
SEWER SERVICE STUDY FOR URBAN GROWTH AREA

Sheet Title: URBAN GROWTH AREA (UGA)
WEST SERVICE AREA
PROPOSED SEWER PLAN

Figure No. 9

Date: SEPTEMBER 2003

Scale: 1" = 1000'

MV-UGA-…G9.dwg 09/12/03 GUNDY

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
North UGA																		
Thillberg Trunk																		
N15-A (1)	155.0	153.9	147.0	143.0	9.5	1000	8	0.004	101	0.00	0.00	0.111	0.101	3.20	0.43	0.49	87.59	
N15-A (2)	153.9	112.0	143.0	104.0	9.5	1000	8	0.039		0.11	0.10	0.000	0.000	3.20	0.43	1.54	28.05	
N15-A (3)	112.0	85.0	104.0	77.0	8.0	1000	8	0.027		0.11	0.10	0.000	0.000	3.20	0.43	1.28	33.71	
N15-A (4)	85.0	58.0	77.0	50.0	8.0	1300	8	0.0208		0.11	0.10	0.000	0.000	3.20	0.43	1.13	38.44	
N15-C (1)	58.0	57.3	50.0	46.0	9.7	700	10	0.0057	64	0.11	0.10	0.070	0.064	3.07	0.69	1.07	64.15	
N15-C (2)	57.3	56.4	46.0	42.0	12.9	1000	10	0.004		0.18	0.16	0.000	0.000	3.07	0.69	0.90	76.67	
N15-C (3)	56.4	55.8	42.0	40.0	15.1	600	10	0.0033		0.18	0.16	0.000	0.000	3.07	0.69	0.82	83.99	
Swan Road Trunk																		
N15-B (1)	134.1	122.2	126.1	112	9.1	1100	8	0.0128	130	0.00	0.00	0.143	0.130	3.13	0.55	0.88	62.21	
N15-B (2)	122.2	117.5	112	102	12.9	1100	8	0.0091		0.14	0.13	0.000	0.000	3.13	0.55	0.74	73.87	
N15-B (3)	117.5	90.0	102.0	82.0	11.8	1100	8	0.0182		0.14	0.13	0.000	0.000	3.13	0.55	1.05	52.24	
N15-B (4)	90.0	67.6	82.0	59.6	8.0	1000	8	0.0224		0.14	0.13	0.000	0.000	3.13	0.55	1.17	47.06	
N15-B (5)	67.6	57.2	59.6	49.2	8.0	1000	8	0.0104		0.14	0.13	0.000	0.000	3.13	0.55	0.80	69.07	
N15-B (6)	158.7	122.2	150.7	114.2	8.0	800	8	0.0456	2	0.00	0.00	0.002	0.002	4.22	0.01	1.67	0.64	
N15-D (1)	134.1	122.9	126.1	114.9	8.0	1000	10	0.0112	57	0.14	0.13	0.063	0.057	3.04	0.77	1.50	51.62	
N15-D (2)	122.9	44.0	114.9	36.0	8.0	1250	10	0.0631		0.21	0.19	0.000	0.000	3.04	0.77	3.56	21.74	
N15-D (3)	56.0	43.0	40.0	30.0	14.5	1000	10	0.01		0.21	0.19	0.000	0.000	3.04	0.77	1.42	54.63	
N15-D (4)	43.0	42.0	30.0	26.0	14.5	800	12	0.005		0.21	0.19	0.000	0.000	3.04	0.77	1.63	47.51	
N15-D (5)	130.0	105.0	122.0	97.0	8.0	1000	8	0.025	2	0.00	0.00	0.002	0.002	4.22	0.01	1.23	0.86	
N16 (1)	134.7	118.0	126.7	110.0	8.0	1000	8	0.0167	47	0.00	0.00	0.052	0.047	3.40	0.21	1.01	20.89	
N16 (2)	118.0	70.0	110.0	62.0	8.0	800	8	0.06		0.05	0.05	0.000	0.000	3.40	0.21	1.91	11.02	
N9-F (1)	152.0	150.0	144.0	142.0	8.0	600	8	0.0033	25	0.00	0.00	0.028	0.025	3.56	0.31	0.45	68.75	
N9-F (2)	151.0	130.0	143.0	122.0	8.0	1100	8	0.0191		0.08	0.07	0.000	0.000	3.29	0.32	1.08	29.22	
West UGA																		
Dunbar/Sunset Trunk																		
W2-A (1)	23.7	22.3	15.7	14.3	8.0	1000	8	0.0014	47	0.00	0.00	0.052	0.047	3.40	0.21	0.29	72.14	1/2 of W2-A basin
W2-A (2)	22.3	20.9	14.3	12.9	8.0	1000	8	0.0014		0.05	0.05	0.000	0.000	3.40	0.21	0.29	72.32	
W2-A (3)	20.9	19.5	12.9	11.0	8.3	1000	8	0.0019		0.05	0.05	0.000	0.000	3.40	0.21	0.34	62.08	
W2-A (4)	19.5	19.1	11.0	9.1	9.3	1000	8	0.0019		0.05	0.05	0.000	0.000	3.40	0.21	0.34	62.14	1/2 of W2-A basin
W2-A (5)	19.1	18.7	9.1	7.7	10.5	1000	8	0.0014		0.05	0.05	0.000	0.000	3.40	0.21	0.29	73.54	
W2-A (6)	18.7	18.4	7.7	6.4	11.5	1000	8	0.0014		0.05	0.05	0.000	0.000	3.40	0.21	0.29	73.54	
W2-A (7)	20.7	19.9	12.7	11.4	8.3	1100	8	0.0012	2	0.00	0.00	0.002	0.002	4.22	0.01	0.27	3.96	
Cottonwood/Mc Lean Trunk																		

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
W2-B (1)	20.6	17.4	12.6	9.4	8.0	800	8	0.004	36	0.00	0.00	0.040	0.036	3.47	0.16	0.49	33.28	1/2 of W2-B basin
W2-B (2)	17.4	20.2	9.4	8.2	10.0	800	8	0.0015		0.04	0.04	0.000	0.000	3.47	0.16	0.30	54.34	
W2-B (3)	20.2	21.0	8.2	7.0	13.0	800	8	0.0015		0.04	0.04	0.000	0.000	3.47	0.16	0.30	54.34	
W2-B (4)	21.0	23.4	7.0	5.1	16.2	800	8	0.0024		0.04	0.04	0.000	0.000	3.47	0.16	0.38	43.19	
W2-B (5)	23.4	22.6	5.1	4.6	18.2	800	8	0.0006		0.04	0.04	0.000	0.000	3.47	0.16	0.20	84.19	1/2 of W2-B basin
W2-B (6)	22.6	18.3	4.6	3.3	16.5	500	8	0.0026		0.04	0.04	0.000	0.000	3.47	0.16	0.40	41.28	
W2-B (7)	19.0	23.4	11.0	5.4	13.0	500	8	0.0112	2	0.00	0.00	0.002	0.002	4.22	0.01	0.83	1.29	
South UGA																		
Little Mountain/Blackburn Trunk																		
S5-B (1)	372.9	363.0	364.9	355.0	8.0	1000	10	0.0099	123	0.00	0.00	0.135	0.123	3.15	0.52	1.41	37.07	1/3 of S5-B basin
S5-B (2)	363.0	322.5	355.0	314.5	8.0	1000	10	0.0405		0.14	0.12	0.000	0.000	3.15	0.52	2.85	18.33	2/3 of S5-B basin
S5-B (3)	322.5	260.0	314.5	252.0	8.0	990	10	0.0631		0.14	0.12	0.000	0.000	3.15	0.52	3.56	14.68	
S5-B (4)	310.0	260.0	302.0	252.0	8.0	820	10	0.061		0.14	0.12	0.000	0.000	3.15	0.52	3.50	14.94	
S4-A2 (1)	260.0	194.9	252.0	186.9	8.0	1000	10	0.0651	91	0.14	0.12	0.100	0.091	3.00	1.15	3.61	31.83	
S4-A2 (2)	194.9	175.0	186.9	167.0	8.0	600	10	0.0332		0.24	0.21	0.000	0.000	3.00	0.88	2.58	34.04	
S3-A2 (1)	175.0	146.0	167.0	138.0	8.0	1100	10	0.0264	167	0.24	0.21	0.184	0.167	2.85	1.51	2.30	65.51	exist. SS present
Cedar Hills Dr. Trunk																		
S4-A2 (5)	294.0	265.0	286.0	257.0	8.0	550	8	0.0527	4	0.00	0.00	0.004	0.004	4.04	0.02	1.79	1.15	
S4-A2 (4)	265.0	243.0	257.0	235.0	8.0	600	8	0.0367		0.00	0.00	0.000	0.000	4.04	0.02	1.50	1.37	
S4-A2 (3)	294.0	197.0	286.0	189.0	8.0	950	8	0.1021	4	0.00	0.00	0.004	0.004	4.04	0.02	2.50	0.82	
S4-A2 (6)	197.0	194.9	190.0	186.9	7.5	800	8	0.0039		0.00	0.00	0.000	0.000	4.04	0.02	0.49	4.23	
S4-A1 (1)	155.0	150.0	147.0	137.0	10.5	1000	10	0.01	66	0.24	0.21	0.073	0.066	2.93	1.13	1.42	79.75	
S4-A1 (2)	150.0	125.0	137.0	117.0	10.5	1000	10	0.02		0.31	0.28	0.000	0.000	2.93	1.13	2.00	56.39	
S4-A1 (3)	125.0	113.5	117.0	105.5	8.0	1000	10	0.0115		0.31	0.28	0.000	0.000	2.93	1.13	1.52	74.36	
S3-A2 (2)	113.5	80.0	105.5	72.0	8.0	500	8	0.067	167	0.31	0.28	0.184	0.167	2.81	1.75	2.02	86.47	exist. SS present
S3-A2 (3)	80.0	37.6	72.0	29.6	8.0	650	8	0.0652		0.49	0.45	0.000	0.000	2.81	1.75	1.99	87.63	
Blodgett/Anderson Trunk																		
S3-A1 (1)	37.6	20.0	29.6	12.0	8.0	1200	10	0.0147	67	0.31	0.28	0.074	0.067	2.88	1.38	1.71	80.47	
S3-A1 (2)	20.0	18.0	12.0	-1.0	13.5	1000	10	0.013		0.38	0.35	0.000	0.000	2.88	1.38	1.61	85.47	
S3-A1 (3)	65.3	30.0	57.3	22.0	8.0	1700	10	0.0208	2	0.00	0.00	0.002	0.002	4.22	0.01	2.04	0.52	
Cedardale Rd Trunk																		
S3-B2 (1)	21.0	18.0	13.0	10.0	8.0	800	8	0.0038	84	0.00	0.00	0.092	0.084	3.25	0.36	0.48	75.89	
S3-B2 (2)	18.0	16.0	10.0	7.0	8.5	900	8	0.0033		0.09	0.08	0.000	0.000	3.25	0.36	0.45	80.50	
Montpilar/Skyridge Trunk																		
S4-B (1)	340.0	300.0	332.0	292.0	8.0	1400	8	0.0286	124	0.00	0.00	0.136	0.124	3.14	0.52	1.32	39.73	
S4-B (3)	300.0	200.0	292.0	192.0	8.0	1000	8	0.1		0.14	0.12	0.000	0.000	3.14	0.52	2.47	21.24	
S4-B (4)	173.0	96.0	165.0	88.0	8.0	1200	8	0.0642		0.14	0.12	0.000	0.000	3.14	0.52	1.98	26.51	

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
S4-B (5)	173.0	96.0	165.0	88.0	8.0	1100	8	0.07		0.14	0.12	0.000	0.000	3.14	0.52	2.07	25.38	
S4-B (6)	82.0	38.0	74.0	30.0	8.0	1200	8	0.0367		0.14	0.12	0.000	0.000	3.14	0.52	1.50	35.07	
S4-B (7)	50.0	38.3	42.0	30.3	8.0	1500	8	0.0078		0.14	0.12	0.000	0.000	3.14	0.52	0.69	76.04	
S4-B (8)	38.3	38.0	30.3	22.0	12.0	1300	8	0.0064		0.14	0.12	0.000	0.000	3.14	0.52	0.62	84.05	
S4-B (2)	290.0	245.0	282.0	237.0	8.0	1800	8	0.025	2	0.00	0.00	0.002	0.002	4.22	0.01	1.23	0.86	
East Hickox Road/PamelaTrunk																		
S4-C (1)	275.0	225.0	267.0	217.0	8.0	700	8	0.0714	136	0.00	0.00	0.150	0.136	3.12	0.57	2.09	27.49	1/2 of S4-C basin
S4-C (2)	225.0	115.0	217.0	107.0	8.0	1000	8	0.11		0.15	0.14	0.000	0.000	3.12	0.57	2.59	22.16	
S4-C (3)	115.0	65.0	107.0	57.0	8.0	1000	8	0.05		0.15	0.14	0.000	0.000	3.12	0.57	1.75	32.86	
S4-C (4)	65.0	58.0	57.0	50.0	8.0	550	8	0.0127		0.15	0.14	0.000	0.000	3.12	0.57	0.88	65.13	
S4-C (5)	220.0	176.0	212.0	168.0	8.0	700	8	0.0629	1	0.00	0.00	0.001	0.001	4.40	0.01	1.96	0.28	
S4-C (6)	58.0	65.0	50.0	41.0	16.0	1250	8	0.0072	137	0.00	0.00	0.151	0.137	3.12	0.58	0.66	87.20	1/2 of S4-C basin
S4-C (7)	70.0	30.0	41.0	22.0	18.5	1350	8	0.0141		0.15	0.14	0.000	0.000	3.12	0.57	0.93	61.94	
Sahale Drive																		
S4-C (8)	190.0	150.0	182.0	142.0	8.0	950	8	0.0421	2	0.00	0.00	0.002	0.002	4.22	0.01	1.60	0.66	
S4-C (9)	150.0	70.0	142.0	62.0	8.0	950	8	0.0842		0.00	0.00	0.000	0.000	4.22	0.01	2.27	0.47	
Cedardale/Dale Way Trunk																		
S3-C (1)	38.0	16.0	22.0	8.0	12.0	1300	12	0.0108	93	0.23	0.21	0.102	0.093	2.91	1.20	2.39	50.35	2/3 of S3-C basin
S3-C (2)	16.0	14.7	8.0	0.7	11.0	1000	12	0.0073		0.33	0.30	0.000	0.000	2.91	1.20	1.97	61.15	
S3-C (3)	14.7	12.0	0.7	-6.0	16.0	900	12	0.0074		0.33	0.30	0.000	0.000	2.91	1.20	1.99	60.56	
S3-C (4)	30.0	17.0	22.0	9.0	8.0	1000	10	0.013	93	0.15	0.14	0.102	0.093	2.98	0.94	1.61	57.93	1/3 of S3-C basin
S3-C (5)	17.0	13.5	9.0	3.5	9.0	1200	12	0.0046		0.25	0.23	0.000	0.000	2.98	0.94	1.56	59.99	
S3-C (6)	13.5	12.0	3.9	-6.0	13.8	1100	12	0.009		0.25	0.23	0.000	0.000	2.98	0.94	2.18	42.81	
S1-D (1)	12.0		4.0	-8.0	8.0	1100	15	0.005	141	0.48	0.44	0.155	0.141	2.74	2.22	2.95	75.05	no contour data
S1-D (2)					8.0	800	10	0.005	141	0.00	0.00	0.155	0.141	3.11	0.59	1.00	59.29	no contour data
S1-D (3)					8.0	1150	10	0.005		0.16	0.14	0.000	0.000	3.11	0.59	1.00	59.29	no contour data
East UGA																		
E-13	500.0	494.0	492.0	484.0	9.0	4500	8	0.0018	65	0.00	0.00	0.071	0.065	3.31	0.28	0.33	86.46	
Andal Road Trunk																		
E-11 (1)	496.0	476.0	484.0	458.0	15.0	3000	8	0.0087	90	0.07	0.06	0.099	0.090	3.09	0.65	0.73	88.96	
E-11 (2)	541.0	484.0	533.0	476.0	8.0	2300	8	0.0248		0.10	0.09	0.000	0.000	3.23	0.39	1.23	31.68	
E-15	484.0	350.0	476.0	342.0	8.0	2500	8	0.0536	40	0.17	0.15	0.044	0.040	3.03	0.80	1.81	44.39	

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
E-12	484.0	400.0	476.0	392.0	8.0	3000	8	0.028	104	0.00	0.00	0.114	0.104	3.19	0.45	1.31	34.14	
Mountain View Road Trunk																		
E-14(1)	483.0	231.0	475.0	223.0	8.0	2000	8	0.126	86	0.00	0.00	0.095	0.086	3.24	0.37	2.77	13.46	
E-14(2)	231.0	100.0	223.0	92.0	8.0	2200	8	0.0595		0.21	0.19	0.000	0.000	3.03	0.79	1.91	41.20	
State Highway 9 Trunk																		
E-8 (2)	337.0	112.0	329.0	104.0	8.0	1900	8	0.1184	176	0.21	0.19	0.194	0.176	2.86	1.47	2.69	54.57	
E-8 (3)	112.0	100.0	104.0	92.0	8.0	4800	15	0.0025		0.41	0.37	0.000	0.000	2.86	1.47	2.09	70.26	
E-8 (1)	145.0	112.0	137.0	104.0	8.0	2300	10	0.0143	2	0	0.00	0.002	0.002	4.22	0.22	1.70	13.24	
E-8 (4)	163.9	158.0	155.9	140.0	13.0	2000	15	0.008		0.62	0.56	0.000	0.000	2.86	2.22	3.72	59.61	
Burlingame Road / Big Lake Road Trunk																		
E-3 (1)	391.0	393.0	383.0	381.0	10.0	1200	8	0.0017	42	0.00	0.00	0.046	0.042	3.43	0.19	0.32	59.62	
E-3 (2)	393.0	395.0	385.0	383.0	10.0	1200	8	0.0017		0.05	0.04	0.000	0.000	3.43	0.19	0.32	59.62	
E-1 (1)	395.0	317.7	383.0	307.0	11.3	1000	8	0.076	209	0.05	0.04	0.230	0.209	2.96	1.02	2.15	47.33	
E-1 (2)	317.7	240.3	307.0	231.0	10.0	1000	8	0.076		0.28	0.25	0.000	0.000	2.96	1.02	2.15	47.33	
E-1 (3)	240.3	163.0	231.0	155.0	8.7	1000	8	0.076		0.28	0.25	0.000	0.000	2.96	1.02	2.15	47.33	
E-6	391.0	355.0	383.0	347.0	8.0	1100	8	0.0327	28	0.00	0.00	0.031	0.028	3.53	0.13	1.41	9.18	
Kato Lane/Mountainview Road Trunk																		
E-9 (1)	522.0	500.0	514.0	492.0	8.0	1000	8	0.022	140	0.00	0.00	0.154	0.140	3.11	0.59	1.16	50.91	
E-9 (2)	500.0	340.0	492.0	332.0	8.0	1250	8	0.128		0.15	0.14	0.000	0.000	3.11	0.59	2.79	21.10	
E-9 (3)	340.0	230.0	332.0	222.0	8.0	1250	8	0.088		0.15	0.14	0.000	0.000	3.11	0.59	2.32	25.45	
E-7	235.0	207.0	227.0	199.0	8.0	800	8	0.035	63	0.15	0.14	0.069	0.063	3.02	0.84	1.46	57.18	
E-4 (1)	305.0	207.0	297.0	199.0	8.0	1200	8	0.0817	30	0.00	0.00	0.033	0.030	3.51	0.14	2.23	6.20	1/3 of E-4 basin
E-4 (2)	207.0	195.0	199.0	172.5	15.3	1100	8	0.0241	60	0.22	0.20	0.066	0.060	2.95	1.06	1.21	87.82	2/3 of E-4 basin
E-4 (3)	195.0	150.0	172.5	142.0	15.3	1000	8	0.0305		0.29	0.26	0.000	0.000	2.95	1.06	1.36	78.05	
E-4 (4)	150.0	158.0	142.0	139.0	13.5	1100	12	0.0027		0.29	0.26	0.000	0.000	2.95	1.06	1.20	88.53	
State Highway 9 Trunk, cont'd																		
E-1 (4)	163.0	117.0	146.0	109.0	12.5	2300	15	0.0161		0.89	0.56	0.000	0.000	2.75	2.43	5.30	45.97	
E-1 (5)	117.0	100.0	109.0	85.0	11.5	3600	15	0.0067		0.89	0.56	0.000	0.000	2.75	2.43	3.41	71.42	
N10-C (1)	100.0	43.0	85.0	22.0	18.0	4000	15	0.0158	37	1.19	1.08	0.041	0.037	2.57	4.11	5.24	78.43	
E-2	382.0	379.0	374.0	371.0	8.0	1500	8	0.002	35	0.00	0.00	0.039	0.035	3.47	0.16	0.35	45.83	
N10-H	379.0	360.0	371.0	352.0	8.0	1750	8	0.0109	55	0.04	0.04	0.061	0.055	3.23	0.39	0.81	47.86	
N10-E	360.0	300.0	352.0	292.0	8.0	1000	8	0.06	44	0.099	0.090	0.048	0.044	3.12	0.57	1.91	29.58	
N10-C (2)	300.0	100.0	292.0	92.0	8.0	1400	8	0.1429	37	0.14	0.04	0.041	0.037	3.29	0.41	2.95	14.05	
E-5 (1)	394.0	391.3	386.0	383.3	8.0	1000	8	0.0027	57	0.00	0.00	0.063	0.057	3.35	0.25	0.41	62.45	
E-5 (2)	391.3	388.6	383.3	380.6	8.0	1000	8	0.0027		0.06	0.06	0.000	0.000	3.35	0.25	0.41	62.45	
E-5 (3)	388.6	385.9	380.6	377.9	8.0	1000	8	0.0027		0.06	0.06	0.000	0.000	3.35	0.25	0.41	62.45	

Drainage Segment ID	1 Upstream MH at Grade Elevation	2 Down-stream MH at Grade Elevation	3 Up-stream MH Invert Elevation	4 Down-stream MH Invert Elevation	5 Average Sewer Depth (ft)	6 Length (ft)	7 Dia-meter (in)	8 Slope	9 Service Area (ac)	10 Upstream Infilt (mgd)	11 Upstream Avg San (mgd)	12 Infilt (mgd)	13 Avg San Flow (mgd)	14 Peak Factor	15 Peak Flow (mgd)	16 Avail Capacity (mgd)	17 Percent Utilized (%)	Notes
E-10		2.7							78	0.00	0.00	0.086	0.078					no individual collector pipe

APPENDIX B

Example of TABULA Cost Output and Additional Information

Appendix B – Example of TABULA Cost Output and Additional Information

The preceding examples are for 8-inch sewer pipe at different depths.

For additional information or to download the TABULA software, go to http://www.bugbytes.com/tabula/.

Cost Calculations for Pipe: **8" - 8' depth**

Project year: 2003

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2003
Length: 1000 ft
Conduit Type: Gravity Sewer
Depth of Cover: 8 ft
Trench Backfill Type: Native
Manhole Spacing: Average (500 ft)
Existing Utilities: Average
Dewatering: Minimal
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in.

Geometry

Outer Diameter	0.875 ft
Trench Width	3.64 ft
Excavation Depth	9.88 ft
Complete Surface Rest. Width	5.64 ft

Unit Costs (Basis 1999)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	1,330	CY	10.00	13,300
Backfill	943	CY	5.00	4,720
Complete Pavement Restoration	626	SY	50.00	31,300
Trench Safety	19,750	SF	0.50	9,880
Spoil Load and Haul	387	CY	10.00	3,870
Pipe Unit Material Cost	1,000	lf	10.00	10,000
Pipe Installation	1,000	lf	10.00	10,000
Place Pipe Zone Fill	365	CY	25.00	9,130
Manholes	2	MH	3,000.00	6,000
Existing Utilities	1,000	lf	20.00	20,000
Dewatering	1,000	lf	20.00	20,000
Traffic Control	1,000	lf	5.00	5,000

Year 1999 subtotal	143,000
Mobilization/Demobilization at 10%	1.10
1999 to 2003	1.13
Effective Multiplier	1.24
Subtotal	177,000

Total: $177,000

Cost Calculations for Pipe: **8" - 10' depth**

Project year: 2003

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

<u>Assumptions</u>

Construction Year: 2003
Length: 1000 ft
Conduit Type: Gravity Sewer
Depth of Cover: 10 ft
Trench Backfill Type: Native
Manhole Spacing: Average (500 ft)
Existing Utilities: Average
Dewatering: Minimal
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in.

<u>Geometry</u>

Outer Diameter	0.875 ft
Trench Width	3.64 ft
Excavation Depth	11.9 ft
Complete Surface Rest. Width	5.64 ft

<u>Unit Costs</u> (Basis 1999)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	1,600	CY	10.00	16,000
Backfill	1,213	CY	5.00	6,060
Complete Pavement Restoration	626	SY	50.00	31,300
Trench Safety	23,750	SF	0.50	11,900
Spoil Load and Haul	387	CY	10.00	3,870
Pipe Unit Material Cost	1,000	lf	10.00	10,000
Pipe Installation	1,000	lf	10.00	10,000
Place Pipe Zone Fill	365	CY	25.00	9,130
Manholes	2	MH	3,000.00	6,000
Existing Utilities	1,000	lf	20.00	20,000
Dewatering	1,000	lf	20.00	20,000
Traffic Control	1,000	lf	5.00	5,000

Year 1999 subtotal	149,000
Mobilization/Demobilization at 10%	1.10
1999 to 2003	1.13
Effective Multiplier	1.24
Subtotal	185,000

Total: $185,000

Cost Calculations for Pipe: **8" - 12' depth**

Project year: 2003

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2003
Length: 1000 ft
Conduit Type: Gravity Sewer
Depth of Cover: 12 ft
Trench Backfill Type: Native
Manhole Spacing: Average (500 ft)
Existing Utilities: Average
Dewatering: Minimal
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in.

Geometry

Outer Diameter	0.875 ft
Trench Width	3.64 ft
Excavation Depth	13.9 ft
Complete Surface Rest. Width	5.64 ft

Unit Costs (Basis 1999)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	1,869	CY	10.00	18,700
Backfill	1,482	CY	5.00	7,410
Complete Pavement Restoration	626	SY	50.00	31,300
Trench Safety	27,750	SF	0.50	13,900
Spoil Load and Haul	387	CY	10.00	3,870
Pipe Unit Material Cost	1,000	lf	10.00	10,000
Pipe Installation	1,000	lf	10.00	10,000
Place Pipe Zone Fill	365	CY	25.00	9,130
Manholes	2	MH	3,000.00	6,000
Existing Utilities	1,000	lf	20.00	20,000
Dewatering	1,000	lf	20.00	20,000
Traffic Control	1,000	lf	5.00	5,000

Year 1999 subtotal	155,000
Mobilization/Demobilization at 10%	1.10
1999 to 2003	1.13
Effective Multiplier	1.24
Subtotal	192,000

Total: $192,000

Cost Calculations for Pipe: **8" - 15' depth**

Project year: 2003

The estimated construction cost below, which includes contractor overhead and profit, is for planning purposes only. The output does NOT include contingency, sales tax, or allied costs (design, permitting, construction management, etc.).

Assumptions

Construction Year: 2003
Length: 1000 ft
Conduit Type: Gravity Sewer
Depth of Cover: 15 ft
Trench Backfill Type: Native
Manhole Spacing: Average (500 ft)
Existing Utilities: Average
Dewatering: Minimal
Pavement Restoration: Trench Width
Traffic: Light
Land Acquisition: None
Required Easements: None
Trench Safety: Standard
Pipe Diameter: 8 in.

Geometry

Outer Diameter	0.875 ft
Trench Width	3.64 ft
Excavation Depth	16.9 ft
Complete Surface Rest. Width	5.64 ft

Unit Costs (Basis 1999)

Item	Quantity	Unit	Unit Cost	ItemCost
Excavation	2,273	CY	10.00	22,700
Backfill	1,886	CY	5.00	9,430
Complete Pavement Restoration	626	SY	50.00	31,300
Trench Safety	33,750	SF	0.50	16,900
Spoil Load and Haul	387	CY	10.00	3,870
Pipe Unit Material Cost	1,000	lf	10.00	10,000
Pipe Installation	1,000	lf	10.00	10,000
Place Pipe Zone Fill	365	CY	25.00	9,130
Manholes	2	MH	3,750.00	7,500
Existing Utilities	1,000	lf	20.00	20,000
Dewatering	1,000	lf	20.00	20,000
Traffic Control	1,000	lf	5.00	5,000

Year 1999 subtotal	166,000
Mobilization/Demobilization at 10%	1.10
1999 to 2003	1.13
Effective Multiplier	1.24
Subtotal	205,000

Total: $205,000

APPENDIX C

UGA Cost Estimates

Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost [1]	Taxes (8.8%)	Contingency [2] (40%)	Engineering and Administration Costs [3] (25%)	Project Cost [4]
North UGA										
N15-A (1)	9.5	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	**$ 360,000**
N15-A (2)	9.5	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	**$ 360,000**
N15-A (3)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N15-A (4)	8.0	1300	8	LF	$ 177	$ 230,100	$ 20,249	$ 100,140	$ 87,622	**$ 440,000**
N15-C (1)	9.7	700	10	LF	$ 195	$ 136,500	$ 12,012	$ 59,405	$ 51,979	**$ 260,000**
N15-C (2)	12.9	1000	10	LF	$ 203	$ 203,000	$ 17,864	$ 88,346	$ 77,302	**$ 390,000**
N15-C (3)	15.1	600	10	LF	$ 216	$ 129,600	$ 11,405	$ 56,402	$ 49,352	**$ 250,000**
N15-B (1)	9.1	1100	8	LF	$ 185	$ 203,500	$ 17,908	$ 88,563	$ 77,493	**$ 390,000**
N15-B (2)	12.9	1100	8	LF	$ 192	$ 211,200	$ 18,586	$ 91,914	$ 80,425	**$ 410,000**
N15-B (3)	11.8	1100	8	LF	$ 192	$ 211,200	$ 18,586	$ 91,914	$ 80,425	**$ 410,000**
N15-B (4)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N15-B (5)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N15-B (6)	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
N15-D (1)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
N15-D (2)	8.0	1250	10	LF	$ 187	$ 233,750	$ 20,570	$ 101,728	$ 89,012	**$ 450,000**
N15-D (3)	14.5	1000	10	LF	$ 216	$ 216,000	$ 19,008	$ 94,003	$ 82,253	**$ 420,000**
N15-D (4)	14.5	800	12	LF	$ 238	$ 190,400	$ 16,755	$ 82,862	$ 72,504	**$ 370,000**
N15-D (5)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N16 (1)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
N16 (2)	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
N9-F (1)	8.0	600	8	LF	$ 177	$ 106,200	$ 9,346	$ 46,218	$ 40,441	**$ 210,000**
N9-F (2)	8.0	1100	8	LF	$ 177	$ 194,700	$ 17,134	$ 84,733	$ 74,142	**$ 380,000**
PS 1/N15-B & 3,400 lf FM		390		GPM	$ 2,100	$ 819,000	$ 72,072	$ 356,429	$ 311,875	**$ 1,560,000**
PS 2/N16 & 1,600 lf FM		150		GPM	$ 4,000	$ 600,000	$ 52,800	$ 261,120	$ 228,480	**$ 1,150,000**
PS 3/N15-D & 600 lf FM		875		GPM	$ 700	$ 612,500	$ 53,900	$ 266,560	$ 233,240	**$ 1,170,000**
						$ 6,030,000				**$ 11,580,000**

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Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost [1]	Taxes (8.8%)	Contingency [2] (40%)	Engineering and Administration Costs [3] (25%)	Project Cost [4]
South UGA										
S5-B (1)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
S5-B (2)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
S5-B (3)	8.0	990	10	LF	$ 187	$ 185,130	$ 16,291	$ 80,569	$ 70,498	**$ 360,000**
S5-B (4)	8.0	820	10	LF	$ 187	$ 153,340	$ 13,494	$ 66,734	$ 58,392	**$ 300,000**
S4-A2 (1)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
S4-A2 (2)	8.0	600	10	LF	$ 187	$ 112,200	$ 9,874	$ 48,829	$ 42,726	**$ 220,000**
S4-A2 (3)	7.5	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
S4-A2 (4)	8.0	950	8	LF	$ 177	$ 168,150	$ 14,797	$ 73,179	$ 64,032	**$ 330,000**
S4-A2 (5)	8.0	550	8	LF	$ 177	$ 97,350	$ 8,567	$ 42,367	$ 37,071	**$ 190,000**
S4-A2 (6)	8.0	600	8	LF	$ 177	$ 106,200	$ 9,346	$ 46,218	$ 40,441	**$ 210,000**
S3-A2 (1)	8.0	1100	10	LF	$ 187	$ 205,700	$ 18,102	$ 89,521	$ 78,331	**$ 400,000**
S4-A1 (1)	10.5	1000	10	LF	$ 203	$ 203,000	$ 17,864	$ 88,346	$ 77,302	**$ 390,000**
S4-A1 (2)	10.5	1000	10	LF	$ 203	$ 203,000	$ 17,864	$ 88,346	$ 77,302	**$ 390,000**
S4-A1 (3)	8.0	1000	10	LF	$ 187	$ 187,000	$ 16,456	$ 81,382	$ 71,210	**$ 360,000**
S3-A2 (2)	8.0	500	8	LF	$ 177	$ 88,500	$ 7,788	$ 38,515	$ 33,701	**$ 170,000**
S3-A2 (3)	8.0	650	8	LF	$ 177	$ 115,050	$ 10,124	$ 50,070	$ 43,811	**$ 220,000**
S3-A1 (1)	8.0	1200	10	LF	$ 187	$ 224,400	$ 19,747	$ 97,659	$ 85,452	**$ 430,000**
S3-A1 (2)	13.5	1000	10	LF	$ 216	$ 216,000	$ 19,008	$ 94,003	$ 82,253	**$ 420,000**
S3-A1 (3)	8.0	1700	10	LF	$ 187	$ 317,900	$ 27,975	$ 138,350	$ 121,056	**$ 610,000**
S3-B2 (1)	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	**$ 270,000**
S3-B2 (2)	8.5	900	8	LF	$ 185	$ 166,500	$ 14,652	$ 72,461	$ 63,403	**$ 320,000**
S4-B (1)	8.0	1400	8	LF	$ 177	$ 247,800	$ 21,806	$ 107,843	$ 94,362	**$ 480,000**
S4-B (2)	8.0	1800	8	LF	$ 177	$ 318,600	$ 28,037	$ 138,655	$ 121,323	**$ 610,000**
S4-B (3)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
S4-B (4)	8.0	1200	8	LF	$ 177	$ 212,400	$ 18,691	$ 92,436	$ 80,882	**$ 410,000**
S4-B (5)	8.0	1100	8	LF	$ 177	$ 194,700	$ 17,134	$ 84,733	$ 74,142	**$ 380,000**
S4-B (6)	8.0	1200	8	LF	$ 177	$ 212,400	$ 18,691	$ 92,436	$ 80,882	**$ 410,000**

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Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost (1)	Taxes (8.8%)	Contingency (2) (40%)	Engineering and Administration Costs (3) (25%)	Project Cost (4)
S4-B (7)	8.0	1500	8	LF	$ 177	$ 265,500	$ 23,364	$ 115,546	$ 101,102	**$ 510,000**
S4-B (8)	12.0	1300	8	LF	$ 192	$ 249,600	$ 21,965	$ 108,626	$ 95,048	**$ 480,000**
S4-C (1)	8.0	700	8	LF	$ 177	$ 123,900	$ 10,903	$ 53,921	$ 47,181	**$ 240,000**
S4-C (2)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
S4-C (3)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
S4-C (5)	8.0	700	8	LF	$ 177	$ 123,900	$ 10,903	$ 53,921	$ 47,181	**$ 240,000**
S4-C (4)	8.0	550	8	LF	$ 177	$ 97,350	$ 8,567	$ 42,367	$ 37,071	**$ 190,000**
S4-C (6)	16.0	1250	8	LF	$ 205	$ 256,250	$ 22,550	$ 111,520	$ 97,580	**$ 490,000**
S4-C (7)	18.5	1350	8	LF	$ 177	$ 238,950	$ 21,028	$ 103,991	$ 90,992	**$ 460,000**
S4-C (8)	8.0	950	8	LF	$ 177	$ 168,150	$ 14,797	$ 73,179	$ 64,032	**$ 330,000**
S4-C (9)	8.0	950	8	LF	$ 177	$ 168,150	$ 14,797	$ 73,179	$ 64,032	**$ 330,000**
S3-C (1)	12.0	1300	12	LF	$ 223	$ 289,900	$ 25,511	$ 126,164	$ 110,394	**$ 560,000**
S3-C (2)	11.0	1000	12	LF	$ 223	$ 223,000	$ 19,624	$ 97,050	$ 84,918	**$ 430,000**
S3-C (3)	16.0	900	12	LF	$ 223	$ 200,700	$ 17,662	$ 87,345	$ 76,427	**$ 390,000**
S3-C (4)	8.0	1000	10	LF	$ 203	$ 203,000	$ 17,864	$ 88,346	$ 77,302	**$ 390,000**
S3-C (5)	9.0	1200	12	LF	$ 223	$ 267,600	$ 23,549	$ 116,460	$ 101,902	**$ 510,000**
S3-C (6)	13.8	1100	12	LF	$ 223	$ 245,300	$ 21,586	$ 106,755	$ 93,410	**$ 470,000**
S1-D (1)	8.0	1100	15	LF	$ 270	$ 297,000	$ 26,136	$ 129,254	$ 113,098	**$ 570,000**
S1-D (2)	8.0	800	10	LF	$ 216	$ 172,800	$ 15,206	$ 75,203	$ 65,802	**$ 330,000**
S1-D (3)	8.0	1150	10	LF	$ 216	$ 248,400	$ 21,859	$ 108,104	$ 94,591	**$ 480,000**
PS 1/S4-C & 300 lf FM		770		GPM	$ 780	$ 600,600	$ 52,853	$ 261,381	$ 228,708	**$ 1,150,000**
PS 2/S4-A2 & 1,300 lf FM		150		GPM	$ 5,000	$ 750,000	$ 66,000	$ 326,400	$ 285,600	**$ 1,430,000**
						$ 10,510,000				**$ 20,230,000**
West UGA							$ -	$ -	$ -	
W2-A (1)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
W2-A (2)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**
W2-A (3)	8.3	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	**$ 340,000**

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Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost (1)	Taxes (8.8%)	Contingency (2) (40%)	Engineering and Administration Costs (3) (25%)	Project Cost (4)
W2-A (4)	9.3	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	$ 360,000
W2-A (5)	10.5	1000	8	LF	$ 192	$ 192,000	$ 16,896	$ 83,558	$ 73,114	$ 370,000
W2-A (6)	11.5	1000	8	LF	$ 192	$ 192,000	$ 16,896	$ 83,558	$ 73,114	$ 370,000
W2-A (7)	8.3	1100	8	LF	$ 177	$ 194,700	$ 17,134	$ 84,733	$ 74,142	$ 380,000
W2-B (1)	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	$ 270,000
W2-B (2)	10.0	800	8	LF	$ 185	$ 148,000	$ 13,024	$ 64,410	$ 56,358	$ 290,000
W2-B (3)	13.0	800	8	LF	$ 192	$ 153,600	$ 13,517	$ 66,847	$ 58,491	$ 300,000
W2-B (4)	16.2	800	8	LF	$ 205	$ 164,000	$ 14,432	$ 71,373	$ 62,451	$ 320,000
W2-B (5)	18.2	800	8	LF	$ 205	$ 164,000	$ 14,432	$ 71,373	$ 62,451	$ 320,000
W2-B (6)	16.5	500	8	LF	$ 205	$ 102,500	$ 9,020	$ 44,608	$ 39,032	$ 200,000
W2-B (7)	13.0	500	8	LF	$ 192	$ 96,000	$ 8,448	$ 41,779	$ 36,557	$ 190,000
PS 1/W2-A & 600 lf FM		550		GPM	$ 1,000	$ 550,000	$ 48,400	$ 239,360	$ 209,440	$ 1,050,000
						$ 2,820,000				$ 5,440,000
East UGA										
E-13	9.0	4500	8	LF	$ 185	$ 832,500	$ 73,260	$ 362,304	$ 317,016	$ 1,590,000
E-11 (1)	15.0	3000	8	LF	$ 205	$ 615,000	$ 54,120	$ 267,648	$ 234,192	$ 1,180,000
E-11 (2)	8.0	2300	8	LF	$ 177	$ 407,100	$ 35,825	$ 177,170	$ 155,024	$ 780,000
E-15	8.0	2500	8	LF	$ 177	$ 442,500	$ 38,940	$ 192,576	$ 168,504	$ 850,000
E-14(1)	8.0	2000	8	LF	$ 177	$ 354,000	$ 31,152	$ 154,061	$ 134,803	$ 680,000
E-12	8.0	3000	8	LF	$ 177	$ 531,000	$ 46,728	$ 231,091	$ 202,205	$ 1,020,000
E-14(2)	8.0	2200	8	LF	$ 177	$ 389,400	$ 34,267	$ 169,467	$ 148,284	$ 750,000
E-8 (2)	8.0	1900	8	LF	$ 177	$ 336,300	$ 29,594	$ 146,358	$ 128,063	$ 650,000
E-8 (1)	8.0	2300	10	LF	$ 187	$ 430,100	$ 37,849	$ 187,180	$ 163,782	$ 820,000
E-8 (3)	8.0	4800	15	LF	$ 237	$ 1,137,600	$ 100,109	$ 495,084	$ 433,198	$ 2,170,000
E-8 (4)	13.0	2000	15	LF	$ 255	$ 510,000	$ 44,880	$ 221,952	$ 194,208	$ 980,000
E-3 (1)	10.0	1200	8	LF	$ 185	$ 222,000	$ 19,536	$ 96,614	$ 84,538	$ 430,000
E-3 (2)	10.0	1200	8	LF	$ 185	$ 222,000	$ 19,536	$ 96,614	$ 84,538	$ 430,000

Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost (1)	Taxes (8.8%)	Contingency (2) (40%)	Engineering and Administration Costs (3) (25%)	Project Cost (4)
E-1 (1)	11.3	1000	8	LF	$ 192	$ 192,000	$ 16,896	$ 83,558	$ 73,114	$ 370,000
E-1 (2)	10.0	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	$ 360,000
E-1 (3)	8.7	1000	8	LF	$ 185	$ 185,000	$ 16,280	$ 80,512	$ 70,448	$ 360,000
E-6	8.0	1100	8	LF	$ 177	$ 194,700	$ 17,134	$ 84,733	$ 74,142	$ 380,000
E-9 (1)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	$ 340,000
E-9 (2)	8.0	1250	8	LF	$ 177	$ 221,250	$ 19,470	$ 96,288	$ 84,252	$ 430,000
E-9 (3)	8.0	1250	8	LF	$ 177	$ 221,250	$ 19,470	$ 96,288	$ 84,252	$ 430,000
E-7	8.0	800	8	LF	$ 177	$ 141,600	$ 12,461	$ 61,624	$ 53,921	$ 270,000
E-4 (1)	8.0	1200	8	LF	$ 177	$ 212,400	$ 18,691	$ 92,436	$ 80,882	$ 410,000
E-4 (2)	15.3	1100	8	LF	$ 205	$ 225,500	$ 19,844	$ 98,138	$ 85,870	$ 430,000
E-4 (3)	15.3	1000	8	LF	$ 205	$ 205,000	$ 18,040	$ 89,216	$ 78,064	$ 400,000
E-4 (4)	13.5	1100	12	LF	$ 238	$ 261,800	$ 23,038	$ 113,935	$ 99,693	$ 500,000
E-1 (4)	12.5	2300	15	LF	$ 255	$ 586,500	$ 51,612	$ 255,245	$ 223,339	$ 1,120,000
E-1 (5)	11.5	3600	15	LF	$ 255	$ 918,000	$ 80,784	$ 399,514	$ 349,574	$ 1,750,000
N10-C (1)	18.0	4000	15	LF	$ 270	$ 1,080,000	$ 95,040	$ 470,016	$ 411,264	$ 2,060,000
E-2	8.0	1500	8	LF	$ 177	$ 265,500	$ 23,364	$ 115,546	$ 101,102	$ 510,000
N10-H	8.0	1750	8	LF	$ 177	$ 309,750	$ 27,258	$ 134,803	$ 117,953	$ 590,000
N10-E	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	$ 340,000
N10-C (2)	8.0	1400	8	LF	$ 177	$ 247,800	$ 21,806	$ 107,843	$ 94,362	$ 480,000
E-5 (1)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	$ 340,000
E-5 (2)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	$ 340,000
E-5 (3)	8.0	1000	8	LF	$ 177	$ 177,000	$ 15,576	$ 77,030	$ 67,402	$ 340,000
E-10				LF	$ -	$ -	$ -	$ -	$ -	$ -
PS 1/East UGA &4,300 lf FM		1770		GPM	$ 1,550	$ 2,743,500	$ 241,428	$ 1,193,971	$ 1,044,725	$ 5,230,000
						$ 15,720,000				$ 30,110,000

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Cost Estimate by Basin/Segment

1	2	3	4	5	6	7	8	9	10	11
Description	Average Sewer Depth (ft)	Sewer Length/ Quantity (ft)	Diameter (in)	Unit	Unit Cost	Construction Cost (1)	Taxes (8.8%)	Contingency (2) (40%)	Engineering and Administration Costs (3) (25%)	Project Cost (4)

Notes:
(1) Construction costs include mobilization, excavation, backfill and pipe zone fill, pavement restoration, trench safety, pipe material and installation, manholes, protecting & relocating existing utilities, dewatering, and traffic control.
(2) The construction contingency is an allowance for additional costs not identified in the planning phase and may include utility crossings or unique soil conditions.
(3) Engineering and Administration costs include engineering and design, permitting fees, and City management costs.
(4) Project Costs include taxes (8.8%), contingency (40%), and engineering and administration costs (25%).

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APPENDIX D

Related Documents

City of Mount Vernon
Development Services

910 Cleveland Avenue
Post Office Box 809
Mount Vernon, WA 98273

Phone (360) 336-6214
FAX (360) 336-6283
E-Mail DS@ci.mount-vernon.wa.us
www.ci.mount-vernon.wa.us

August 22, 2003

Chris Parsons
Community, Trade and Economic Development
Growth Management Division
P.O. Box 48300
Olympia, Washington 98504-8300

Dear Chris:

I have enclosed for your information an update to **Mount Vernon's Comprehensive Sewer Plan** adding some additional specificity regarding service to the Urban Growth area. We will be taking this to the Planning Commission and City Council for public hearing later in October.

If you have any questions, please give me a call.

Sincerely,

Elizabeth Sjostrom

Elizabeth Sjostrom
Economic Development Planner

Encl.

City of Mount Vernon

Wastewater Division

1401 Britt Road
Mount Vernon, WA 98273-6511

Phone (360) 336-6219
FAX (360) 424-8749
E-Mail mvwwtp@ci.mount-vernon.wa.us
www.ci.mount-vernon.wa.us

August 25, 2003

Bernard Jones
Department of Ecology
Northwest Regional Office
3190 160th Avenue SE
Bellevue, WA 98008-5452

Subject: Notification of sewer line extensions, including pump stations.

Dear Mr. Jones:

In accordance with WAC 173-240-030(5), the City of Mount Vernon is providing notice of planning for sewer pipe extensions, including pump stations in the urban growth area. The sewer expansion areas are shown in Figure 1 attached. The extension plan is in conformance with the Comprehensive Sewer Plan approved by Ecology on March 4, 2003.

If you have any questions on this matter please contact Walt Enquist at (360) 336-6219.

Sincerely,

Walt Enquist

Walt Enquist
Wastewater Utility Supervisor

Attachments

cc: John Buckley, Pubic Works Director
Elizabeth Sjostrom, Economic Development P
NPDES File

F:\Winword\WALT\COMP-PLA\UGA Amendment 2003\DOE Notifica

7002 0860 0008 7867 1471

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AUG 25 2003
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08/25/03
USPS

Sent To: Bernard Jones
Department of Ecology
Street, Apt. No.; or PO Box No.: Northwest Regional Office
3190 160th Avenue SE
City, State, ZIP+4: Bellevue, WA 98008-5452

PS Form 3800, April 2002 — See Reverse for Instructions

City of MOUNT VERNON
PUBLIC WORKS DEPARTMENT

TECHNICAL MEMO

DATE: July 20, 2016

FROM: Esco Bell, P.E., Public Works Director

SUBJECT: 2016 COMPREHENSIVE PLAN UPDATE AND SANITARY SEWER/WWTP PLANNING

INTRODUCTION:
This memo has been prepared to document that the City is planning for 20-years of growth with its sanitary sewer conveyance systems and Waste Water Treatment Plan through its Capital Facilities Element.

BACKGROUND:
The Growth Management Act (GMA) requires that comprehensive plans include a Capital Facilities Element that addresses the capital facilities needs to adequately support anticipated growth.

The GMA requires that a capital facilities element contain: 1) an inventory of existing capital facilities owned by public entities; 2) a forecast of future needs for such facilities; 3) the proposed locations and capacities of expanded or new facilities; 4) at least a six-year plan that will finance these facilities; and 5) a plan to reassess the land use element if projected funding falls short of meeting existing and expected needs.

The following technical documents are being readopted as part of the City's 2016 update to the Capital Facilities Element of the Comprehensive Plan:

1. Comprehensive Sewer Plan Update dated February 2003 prepared by HDR Engineering
2. Comprehensive Sewer Plan Amendment dated April 2004 prepared by HDR Engineering.
3. Urban Growth Area Sewer Service Study dated October 2003 prepared by HDR Engineering.

OLDER DOCUMENTS STILL VALID:
The above-listed Plans/Studies remain valid planning documents due to the population projections that were used when these documents were originally prepared. Each of these sewer plans used population projections that meet or exceed the projections used to update the City's 2016 Comprehensive Plan.