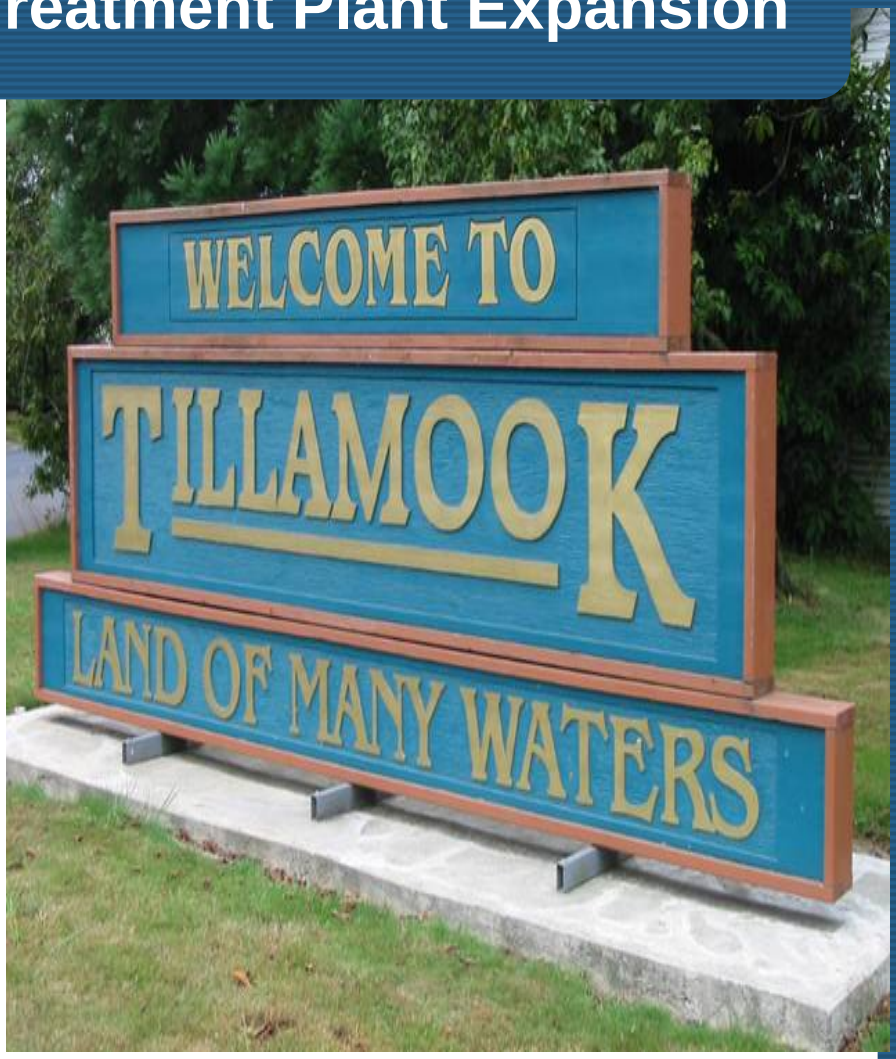
Preliminary Design Report

City of Tillamook
Wastewater Treatment Plant Expansion



Prepared for

City of Tillamook

210 Laurel Avenue
Tillamook, OR 97141

K/J Project No. 0491010.10

February 2006

Kennedy/Jenks Consultants

Engineers & Scientists

City of Tillamook
Wastewater Treatment Plant Expansion

Preliminary Design Report

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February 2006

K/J 0491010.10

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Table of Contents

Introduction

Abbreviations List

Design Criteria Summary

List of Drawings

Process Flow Schematic

Proposed Site Plan

Hydraulic Profile

Preliminary Estimate of Probable Cost

Technical Memorandums

1. Revised Influent Loading Criteria
2. Influent Pump Station
3. Headworks Improvements
4. Liquid Flow Metering
5. Flexible Aeration Basin, Blowers, and Air Piping
6. Secondary Clarification
7. RAS/WAS/Scum Pumping
8. Chlorine Contact Tank and Disinfection Improvements
9. Yard Piping, Hydraulic Control Structures and Plant Utilities
10. Outfall Improvements and COE Permitting
11. Waste Activated Sludge Thickening
12. Anaerobic Digestion and Sludge Storage
13. Solids Processing Building/Blower and Standby Generator Building
14. Septage Receiving Station
15. Electrical, Instrumentation, and SCADA
16. Civil Work, Landscaping and Site Security
17. Front Street Sodium Hypochlorite Dosing Station

Design and Construction Schedule

Geotechnical Report

Table of Contents (cont.)

List of Tables

1.1 Flow and Loading Data (May 2003 to Sept 2005)
1.2 Design Criteria for Influent BOD, TSS, and Ammonia
2.1 Design Flows
5.1 FAB Design Criteria
6.1 Hydraulic Overflow and Solids Loading Rates
7.1 RAS Pumping Capacity
8.1 Anticipated Range of Chlorine Feed Rates
9.1 Pipe Joints
9.2 Wall Penetrations
9.3 Non-Potable Water System Demands
11.1 Sludge Production for the Tillamook WWTP
11.2 WAS Volume at Various Concentrations Following Thickening (gpd)
11.3 Advantages and Disadvantages of Gravity Thickening
11.4 Advantages and Disadvantages of the GBT
11.5 Advantages and Disadvantages of the Rotary Drum Thickener
11.6 Advantages and Disadvantages of the DAF Thickener
11.7 Advantages and Disadvantages of the Centrifuge Thickener
11.8 WAS Thickener Comparison Matrix
11.9 WAS Thickener Design Criteria
12.1 Sludge Production for the Tillamook WWTP
12.2 Digester Design Criteria
12.3 Digested Sludge Storage Volume
13.1 Area Requirements for Thickener Room
13.2 Area Requirements for Digester Appurtenances Room
13.3 Area Requirements for MCC Room
13.4 Area Requirements for the Blower Building
13.5 Code References
13.6 Process Structure Code Requirements
15.1 Electrical Equipment List
15.2 MCC and Electrical Panel List
16.1 New Structure Elevations
17.1 Chlorine Demand

List of Figures

I.1 Existing WWTP Site and North Site
I.2 Proposed Site Plan
I.3 Proposed Site Plan and Aerial Photograph
1.1 Monthly Average Influent Flow
1.2 Monthly Average Influent BOD & TSS Concentration
1.3 Monthly Average BOD and TSS Loading
2.1 Influent Pump Station Plan and Section

Table of Contents (cont.)

3.1 Existing Headworks and Partial Plan of Primary Clarifier
4.1 Process Flow Schematic
5.1 Aeration Basin Plan
5.2 Modified Ludzack-Ettinger (MLE) Process
5.3 Typical Membrane Disc Grade Layout
6.1 Secondary Clarifier Plan and Section
7.1 Secondary Clarifier Plan and Section
8.1 Chlorine Contact Tank Plan and Section
8.2 Automated Control System for Chlorination and Dechlorination
9.1 PEFCS Top Plan and Section
10.1 Effluent Outfall Structure
12.1 Anaerobic Digester Conceptual Layout Plans and Section
12.2 Anaerobic Digester Mixing and Heating Schematic
13.1 Solids Processing Building Floor Plan
13.2 Blower Building Floor Plan
14.1 Septage Receiving Station Schematic
16.1 Proposed Site Plan
17.1 Front Street Sodium Hypochlorite Dosing Station Schematic

Introduction

This Preliminary Design Report is intended to serve as a guide document for the final design of the City of Tillamook WWTP Expansion. Major design topics are addressed in Technical Memorandums 1 through 17. In addition to the Technical Memorandums, the following supporting documentation is included:

- Design Criteria Summary
- List of Design Drawings
- Process Flow Schematic
- Proposed Site Plan
- Hydraulic Profile
- Preliminary Estimate of Probable Cost
- Design and Construction Schedule
- Geotechnical Report

Abbreviations used in this report are defined in the Abbreviations List.

The existing treatment plant is located on a three acre parcel at the west end of 4th Street. It is bordered on the west by the Trask River, on the south by a city-owned boat-launch park, on the east by single-family homes, and on the north by a 2.3 acre grass pasture referred to as the North Site. The majority of the major components of the WWTP expansion will be constructed on the North Site, which the City purchased in 2005. The existing site and the North Site are delineated on Figure I.1 – Existing WWTP Site and North Site. Both sites are essentially flat with an average elevation of 13 feet. The 100-year flood elevation is 11.0 feet

The need for the WWTP expansion was identified in the 2005 Wastewater Facility Plan. The project will consist of the following components:

- New influent pump station
- Existing headworks improvements including a new mechanical screen and bypass channel
- New activated sludge process – incorporating:
  - aeration basin with dual tanks
  - selector zones for control of filamentous sludge
  - diffused aeration equipment and blowers
  - hydraulic control features to maximize operational flexibility
- Two new secondary clarifiers with submersible RAS / WAS / scum pumping
- New chlorine contact tank and disinfection improvements
- Reconstructed effluent outfall pipe

- New Solids Handling Facility including:
  - mechanical thickener
  - anaerobic digester
  - septage receiving station
  - conversion of existing basins for sludge storage

These improvements are intended to increase the reliability, hydraulic capacity, and process capacity of the existing facility such that it will meet the needs of the community for approximately 20 years into the future. The site layout will accommodate additional future expansion allowing for roughly double the upgraded capacity. The proposed site layout is shown on the WWTP Expansion Site Plan, Figure I.2. The proposed site plan is shown superimposed on an aerial photograph on the WWTP Expansion Site Plan and Aerial Photograph, Figure I.3.

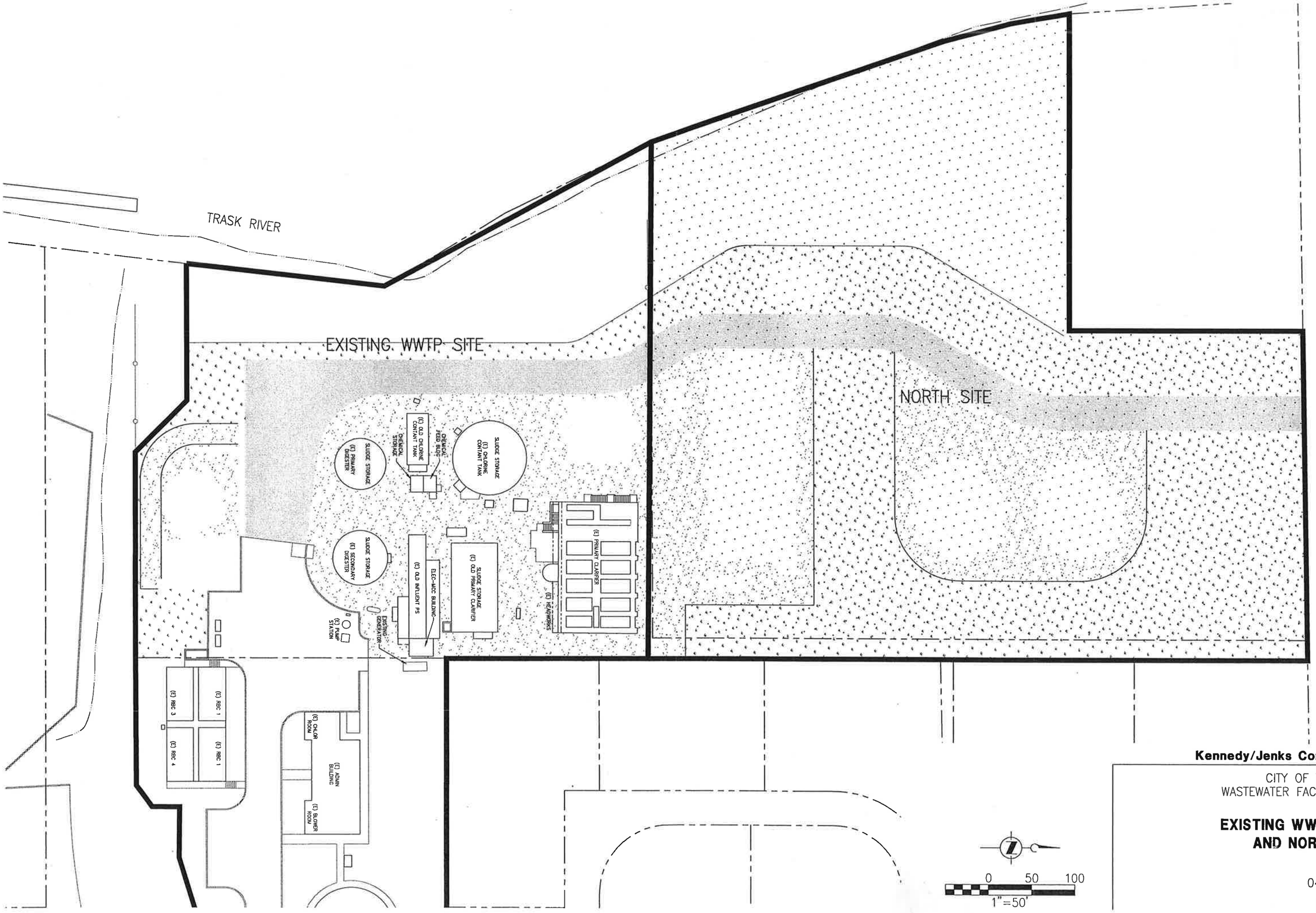
Most of the existing basins, including the south primary clarifier, the primary and secondary digesters, and the secondary clarifiers will be retained and modified to provide sludge storage capacity. The north primary clarifier, built in 1998, will continue to be used for primary treatment. The elevated headworks channel, which is integral to the north primary clarifier, will receive modifications including a mechanical auger screen and bypass channel. The existing Administration and Laboratory Building, built in 1975, will receive minor modifications to convert the blower room into work/storage space. The existing plant must be kept in operation during the construction period.

Raw sewage flows by gravity to the treatment plant through two existing 18-inch concrete pipes. These pipes are well below grade requiring the influent to be pumped prior to treatment. Due to the high degree of inflow and infiltration in the City's collection system, there is an extreme range between average and peak flow. The average dry weather design flow is 0.9 MGD. The peak instantaneous design flow is 10 MGD. Flows in excess of 5.0 MGD will bypass the secondary treatment process.

The WWTP discharges year-round to the Trask River. To protect aquatic species, the new treatment process will be designed to reduce effluent ammonia. Chlorine gas will be used for disinfection and liquid sodium bisulfite will be used for dechlorination. The WWTP is required to meet EPA Class I reliability requirements as it discharges to waters used for shellfish production. Residual solids from the treatment process will be anaerobically digested and land-applied in liquid form.

The Preliminary Estimate of Probable Cost presented in this document is assumed to be accurate within a range of +30% to -15%. The total cost presented includes a 25% Construction Contingency, a 15% multiplier for Contractor Overhead and Profit, and a 5% multiplier for Escalation to Midpoint of Construction. These costs will be refined as final design progresses.

The project will be funded through the DEQ State Revolving Fund, the OECDD Water/Wastewater Loan and Grant Program, and the Community Development Block Grant Program. These programs are all administered through State of Oregon governmental agencies. The start of construction is planned for the spring of 2007.



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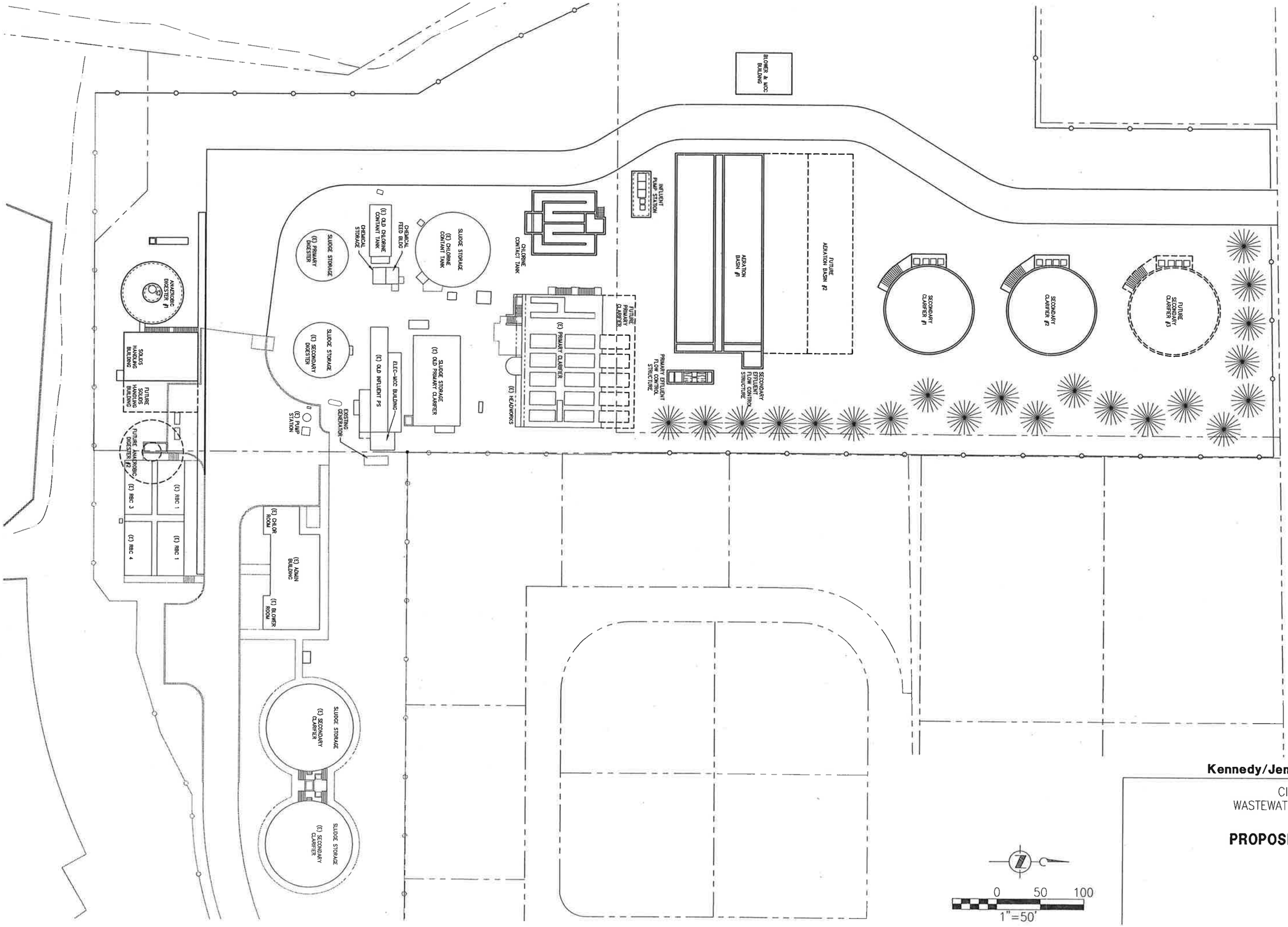
CITY OF TILLAMOOK
WASTEWATER FACILITY PLAN

EXISTING WWTP SITE AND NORTH SITE

0491010.20

FIGURE I.1



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CITY OF TILLAMOOK
WASTEWATER FACILITY PLAN

PROPOSED SITE PLAN

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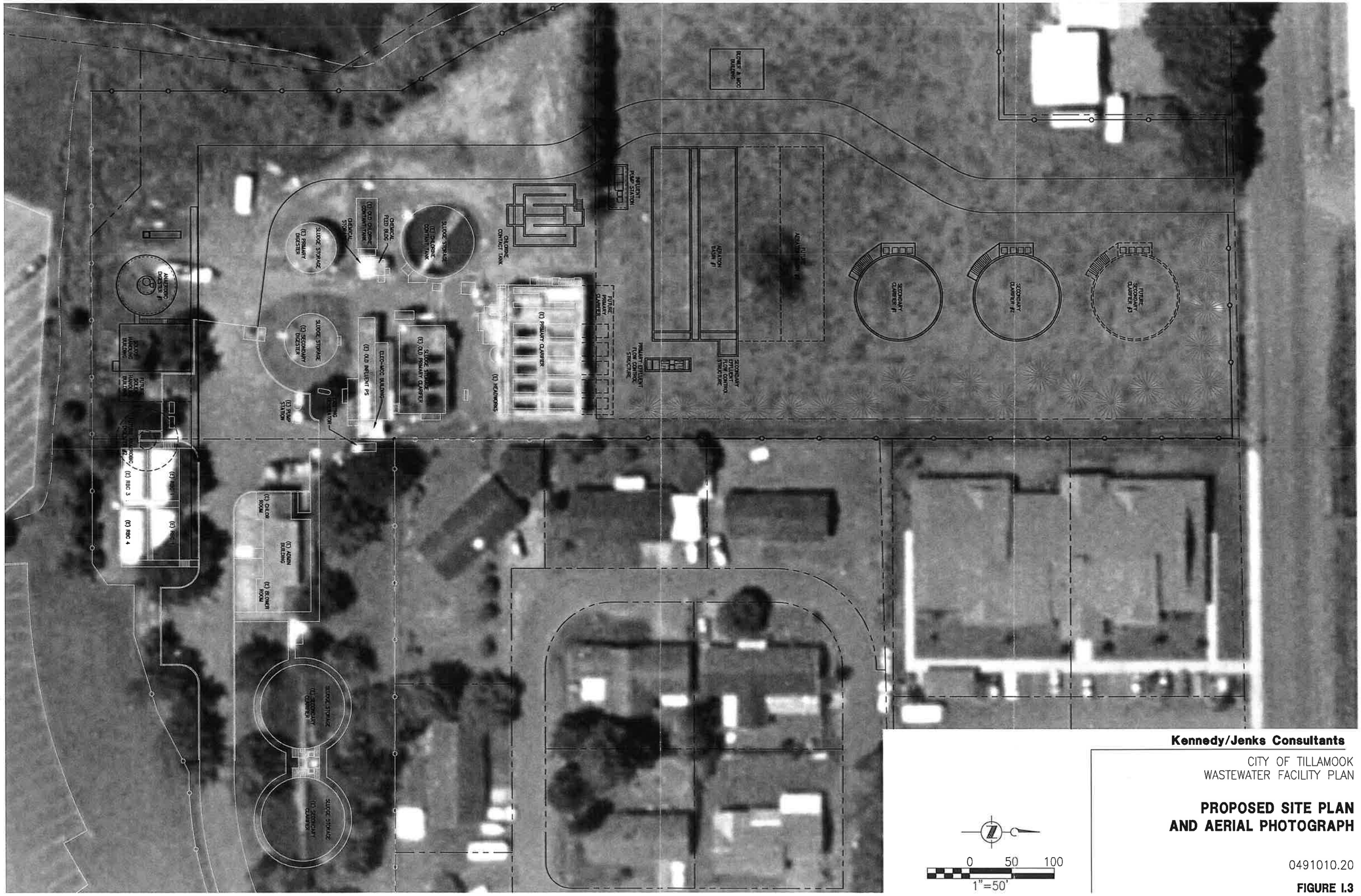
FIGURE I.2



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CITY OF TILLAMOOK
WASTEWATER FACILITY PLAN

PROPOSED SITE PLAN AND AERIAL PHOTOGRAPH

0491010.20

FIGURE I.3

Abbreviations List

Subject: City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Harry Ritter, P.E.

Abbreviation	Description
AFD	adjustable frequency drive
BOD	biochemical oxygen demand
CAP	corrective action plan
CCT	chlorine contact tank
CE	cement epoxy
CIS	cast iron sleeve
CM	cement mortar
City	City of Tillamook
DAF	dissolved air floatation
DEQ	Oregon Department of Environmental Quality
DSL	Oregon Department of State Lands
DMR	Daily Monitoring Report
EPA	Environmental Protection Agency
FAB	flexible aeration basin
FRP	fiberglass reinforced plastic
FE	final effluent
FL	flanged
F/M	food to mass
gal/ft*day	gallons per foot per day
gal/sf*day	gallons per square foot per day
GBT	gravity belt thickener
gpd	gallons per day
gpm	gallons per minute
GRS	gravity flow of raw sewage
HDPE	high density polyethylene
HMI	human-machine interface
hp	horsepower
HRT	hydraulic retention time
HWL	high water level
I-I	infiltration and inflow
IBC	International Building Code
K/J	Kennedy Jenks Consultants
LS	link seal
mA	milliamp
MCC	motor control center
MCRT	mean cell residence time
mg/l	milligrams per liter
MGD	million gallons per day
MJ	mechanical joint

Abbreviations List

City of Tillamook WWTP Expansion
Page 2

Abbreviation	Description
ML	mixed liquor
MLE	Modified Ludzack-Ettinger
MLFSB	Mixed Liquor Flow Splitter Box
MMWWF	maximum month wet weather flow
NFPA	National Fire Protection Association
NPW	nonpotable water
O&M	operation and maintenance
OAR	Oregon Administrative Rule
ORP	oxidation reduction potential
PDF	peak day flow
PE	primary effluent
PEFCS	Primary Effluent Flow Control Structure
PIF	peak instantaneous flow
POJ	push-on joint
PLC	programmable logic controller
ppd	pounds per day
RAS	return activated sludge
RCP	reinforced concrete pipe
RS	raw sewage
SCADA	supervisory control and data acquisition
SCFSB	Secondary Clarifier Flow Splitter Box
SE	secondary effluent
SPGP	Statewide Programmatic General Permit
STEP	septic tank effluent pumping
TDH	total dynamic head
TM	technical memorandum
TSS	total suspended solids
TPUD	Tillamook Public Utilities District
USACE	US Army Corps of Engineers
VFD	variable frequency drive
VS	volatile solids
VTSH	vertical turbine solids handling
WAS	waste activated sludge
WWTP	wastewater treatment plant
ZID	zone of initial dilution

CITY OF TILLAMOOK WWTP EXPANSION DESIGN CRITERIA SUMMARY

CRITERION	START UP VALUE (2007)	DESIGN VALUE (2027)	UNIT
INFLUENT FLOWRATES AND SOLIDS LOADINGS			
Dry Weather Flow			MGD
Average (ADWF)	0.6	0.9	
Maximum Month (MMDWF)	1.5	2.1	
Wet Weather Flow			MGD
Average (AWWF)	1.9	2.5	
Maximum Month (MMWWF)	3.2	4.0	
Peak Day (PDF)[1]	6.0	7.2	
Peak Instantaneous (PIF)[1]	8.5	10.0	
[1] Flow in excess of 5.0 MGD will bypass secondary treatment			
Raw Wastewater Load			ppd
BOD_5 (average month)	1,250	2,100	
BOD_5 (peak month)	1,485	2,480	
BOD_5 (peak week)	2,500	4,200	
TSS (average month)	1,250	2,100	
TSS (peak month)	1,485	2,475	
TSS (peak week)	2,500	4,200	
Ammonia (average month)	135	225	
Ammonia (peak week)	270	450	
Population	**4,400**	7,500	
NPDES Permit Effluent Limitations			
BOD_5 (Dry Weather Months	10 to 20*	10 to 20*	
BOD_5 (Wet Weather Months)	15 to 30*	15 to 30*	
TSS (Dry Weather Months)	10 to 20*	10 to 20*	
TSS (Wet Weather Months)	15 to 30*	15 to 30*	
Ammonia (Dry Weather Months)	10	10	
Ammonia (Wet Weather Months)	15	15	
** Mass Load limits will limit max. concentration*			

EXISTING FACILITIES TO BE DEMOLISHED OR CONVERTED

Comminutor & Parshall Flume	Demolish
Secondary Clarifiers	Convert to Sludge Storage
South Primary Clarifiers	Abandon in-place
Waste Solids Pumping Station	Salvage Equipment
Rotating Biological Contactors	Demolish
RBC Air Blowers	Abandon In-place
Outfall	Demolish
Influent Pump Station	Demolish
Primary Digester	Abandon In-place
Secondary Digester	Convert to Sludge Storage
Supernatant Pump Station	Convert to Septage Pump Station

INFLUENT PUMPING			
Pump Type	Non-clog, Submersible, Variable Speed		
No. Pumps & Motor Size (Ea.)			Hp
In-Service Units	2 @ 45	2 @ 45	
	1 @ 18	1 @ 18	
Standby Units	1 @ 45	1 @ 45	
	1 @ 18	1 @ 18	

CITY OF TILLAMOOK WWTP EXPANSION DESIGN CRITERIA SUMMARY

CRITERION	START UP VALUE (2007)	DESIGN VALUE (2027)	UNIT
HEADWORKS			
Grit Removal			
Type	Vortex (Pista Grit)		
Diameter	8	8	ft
Peak Flow Capacity	6	6	MGD
Fine Screen / Auger			
Type	In-channel Basket Perforated Plate		
Opening Size	¼	¼	inch
Peak Flow Capacity	6	6	MGD
PRIMARY CLARIFICATION			
Primary Clarifiers			
Number	2	2	
Width, each	20	20	ft
Length	60	60	ft
Side water depth	13.7	13.7	ft
Loading Criteria (at PDF)	3,000	3,000	gpd/sqft
ACTIVATED SLUDGE TREATMENT			
Basins			
Number		2	
Average Mean Cell Residence Time		10	days
Organic Loading Rate Range		16-33	lb BOD_5 /d/1000 cu ft
Average Mixed Liquor Suspended Solids Range		1,500 - 3,500	mg/l
Total Selector Zone Volume (3 Zones)		0.12	MG
Oxic Zone Volume		0.66	MG
Total Basin Volume		0.78	MG
Sidewater Depth		20	ft
Basin Width, each		24	ft
Basin Length		108	ft
Total HRT at ADWF (not incl RAS)		20.7	hr
Total HRT at MMWWF (not incl RAS)		4.7	hr
Oxygen Demand Criteria			$lb0_2$/day
Peak Week		5,300	
Peak Month		3,150	
Average Month		2,650	
Aeration Diffusers			
Type	Membrane Disc Fine Bubble		
Air Blowers			
Total Delivery Required (Peak Week)		1,900	scfm
Type	Rotary Lobe (VFD)		psi
Discharge Pressure	9	9.5	psi
Number (includes standby)	3	3	psi
Capacity (each)	950	950	scfm
Size (each)	60	60	Hp
SECONDARY CLARIFICATION			
Clarifier Type	Rapid-Return		
Number	2	2	
Diameter, ea.	50	50	ft
Surface Area - total	3,900	3,900	sq ft
Sidewater Depth	14	14	ft

CITY OF TILLAMOOK WWTP EXPANSION DESIGN CRITERIA SUMMARY

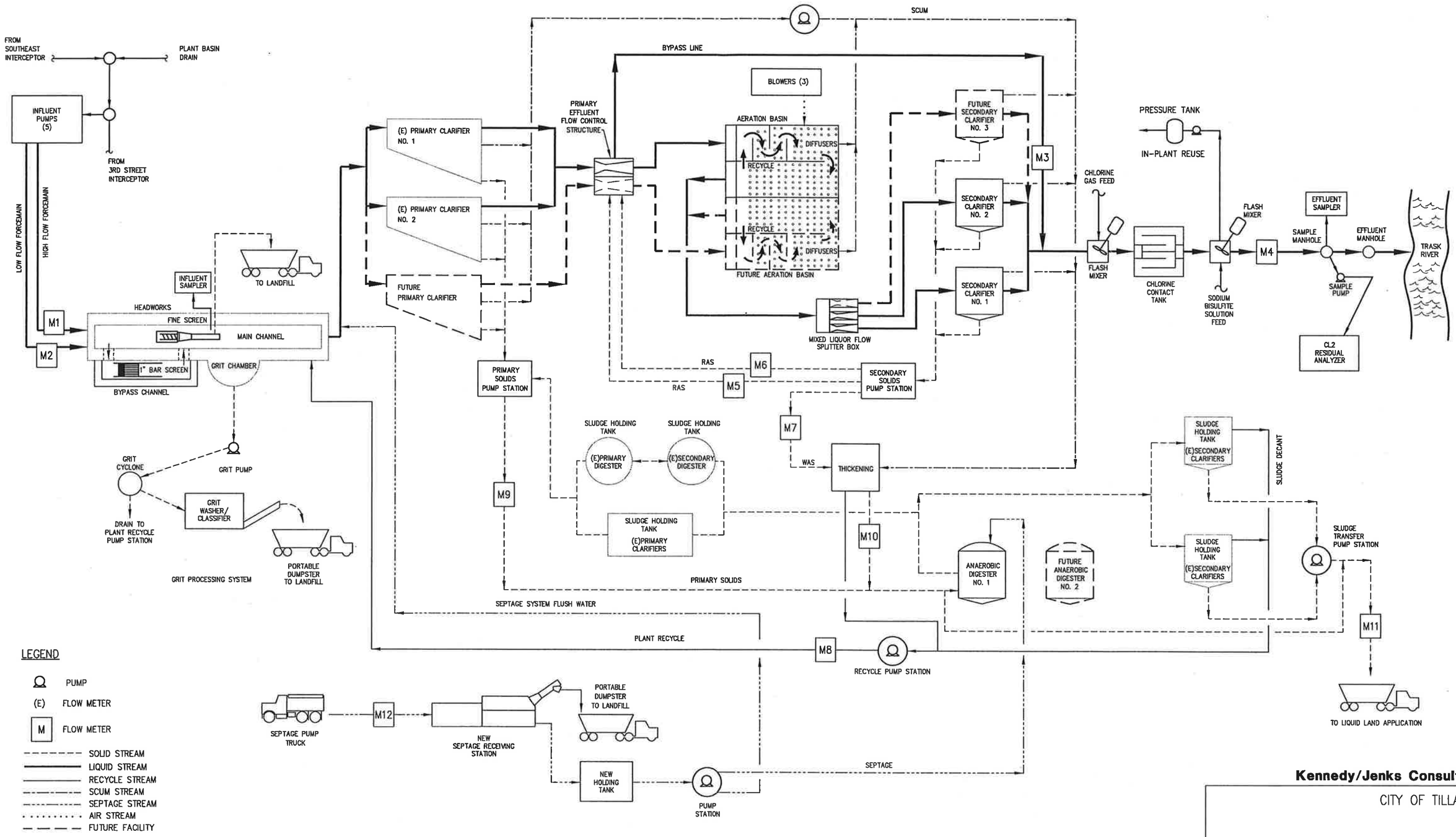
CRITERION	START UP VALUE (2007)	DESIGN VALUE (2027)	UNIT
Overflow Rate *			gpd/sf
ADWF (1 clarifier)		460	
ADWF (2 clarifiers)		NA	
AWWF (1 clarifier)		2,040	
AWWF (2 clarifiers)		1,020	
PDF (1 clarifier)		2,550	
PDF (2 clarifiers)		1,270	
Solids Loading Rate *			ppd/sf
1 clarifier		0.62	
2 clarifiers		0.31	
** N/I RAS. Solids loading rate based on peak month design loading*			
Return Activated Sludge Pumps			
Type	Submersible		
Large Units			
Number (total, 2 clarifiers)	2	2	
Capacity (ea)	1,350	1,350	gpm
Motor Size	15	15	Hp
Small Units			
Number (total, 2 clarifiers)	2	2	
Capacity (ea)	450	450	gpm
Motor Size	5	5	Hp
Waste Activated Sludge Pumps			
Type	Submersible		
Number (total, 2 clarifiers)	2	2	
Capacity (ea)	90	90	gpm
Motor Size	2	2	Hp
Scum Pump			
Type	Submersible Grinder		
Number (total)	2	2	
Motorsize	5	5	Hp
DISINFECTION			
Chlorine Contact Tank			
Number	1	1	
Volume	73,000	73,000	gal
Hydraulic Retention Time at Peak Day Flow	15	15	min.
Chlorination System			
Type	Gas, 150 lb. Cylinders, Remote Injection		
Assumed Dosage Range	3 to 10	3 to 10	mg/l
Control	Flow-Pace & Trim		
Analyzer	Online Cl or ORP		
Dechlorination System			
Type	Liquid Sodium Bisulfite		
Assumed Dosage Range	3 to 6	3 to 6	mg/l
Storage Volume	1,100	1,100	gal
Control	Flow-Pace & Trim		
Analyzer	Online W&T DEOX/2000		

CITY OF TILLAMOOK WWTP EXPANSION DESIGN CRITERIA SUMMARY

CRITERION	START UP VALUE (2007)	DESIGN VALUE (2027)	UNIT
SOLIDS PRODUCTION AND PROCESSING			
Primary Clarifier Effluent Load ⟶ AB			ppd
BOD_5 , Average Month (30% Removal)	875	1,470	
BOD_5 , Peak Month (30% Removal)	1,040	1,733	
TSS, Average Month (60% Removal)	500	840	
TSS, Peak Month (60% Removal)	594	990	
Primary Clarifier Solids Removal ⟶ Waste			ppd
TSS, Average Month	750	1,260	
TSS, Peak Month	891	1,485	
VSS, Average Month	600	1,008	
VSS, Peak Month	713	1,188	
Secondary Clarifier Solids Removal ⟶ Waste			ppd
TSS, Average Month	700	1,176	
TSS, Peak Month	832	1,386	
VSS, Average Month	560	940	
VSS, Peak Month	670	1,110	
Waste Combined Solids ⟶ Digester			
TSS, Average Month	1,450	2,440	ppd
TSS, Peak Month	1,720	2,870	ppd
VSS, Average Month	1,160	1,950	ppd
VSS, Peak Month	1,380	2,300	ppd
Volume, Average Month	4,000	6,800	gpd
Volume, Peak Month	4,800	8,000	gpd
Solids Concentration (Approx.) Prior to Digestion	4.3	4.3	%
Solids Concentration (Approx.) After Digestion (50% VSS Reduction)	2.6	2.6	%
VSS After Digestion, Average Month	580	975	ppd
VSS After Digestion, Peak Month	690	1,150	ppd
TSS After Digestion, Average Month	870	1,440	ppd
TSS After Digestion, Peak Month	1,030	1,740	ppd
Sludge Thickening			
Type	Gravity Belt Thickener		
Frequency of Use			
Days per Week	6	6	days
Hours per Day	4-6	4-6	hours
Feed Rate	50	90	gpm
Size (width)	0.75-1	0.75-1	meter
Final Solids Concentration	6	6	
Anaerobic Digester			
Type	High-Rate, Complete-Mixed		
Number	1	1	
Diameter	34	34	ft
Side Water Depth	20	20	ft
Volume	144,000	144,000	gal
Hydraulic Retention Time	15	15	days
VSS Loading Range (Average)	0.06-0.10	0.06-0.10	lbVSS/cuft-day
Mixing Type	Draft Tube		
Heating Type	Methane / Propane Boiler		
Cover Type	Fixed		
Digested Sludge Storage			
Number of Storage Tanks	5	5	
Volume (Total)	471,000	471,000	gal
Storage Time	128	77	days

City of Tillamook WWTP Expansion
List of Drawings

Sheet	DWG	Title
1	G001	Title Sheet - Region Map and Vicinity Map
2	G002	List of Drawings
3	G003	Abbreviations
4	G004	General Symbols
5	G005	General Notes, Symbols, and Pipe Schedule
6	G006	Plant Process Flow Schematic - Liquid Stream
7	G007	Plant Process Flow Schematic -Solids Stream
8	G008	Flow/Mass Balance
9	G009	Design Data
10	G010	Hydraulic Profile
11	C001	Civil Details 1
12	C002	Civil Details 2
13	C003	Civil Details 3
14	C004	Civil Details 4
15	C101	Existing Facility Demolition Plan (North)
16	C102	Existing Facility Demolition Plan (South)
17	C103	Demolition Details
18	C104	Plant Survey Control, Grading & Paving Plan (North)
19	C105	Plant Survey Control, Grading & Paving Plan (South)
20	C106	Yard Piping Plan (North)
21	C107	Yard Piping Plan (South)
22	C108	Yard Piping Plan Sections and Details 1
23	C109	Yard Piping Plan Sections and Details 2
24	C110	Erosion Control Plan
25	C151	Effluent Outfall Plan and Profile
26	C152	Effluent Outfall Sections and Details
27	L101	Landscape - Irrigation Plan
28	L102	Landscape - Irrigation Details and Legend
29	L103	Landscape - Irrigation Details
30	L104	Landscape - Planting Plan
31	L105	Landscape - Planting Details and Legend
32	L106	Landscape - Planting and Irrigation Specifications
33	S001	Structural General Notes and Standard Details
34	S002	Structural Reinforced Concrete Notes and Standard Details
35	S003	Structural Reinforced Concrete Masonry Unit Notes and Standard Details
36	S004	Stairs, Handrails, Guardrails Details
37	S005	Miscellaneous Structural Details 1
38	S006	Miscellaneous Structural Details 2
39	S007	Miscellaneous Structural Details 3
40	S211	Influent Pumping Station Plan
41	S212	Influent Pumping Station Sections
42	S213	Influent Pumping Station Sections and Details
43	S221	Headworks Modification Plans and Sections 1
44	S222	Headworks Modification Plans and Sections 2
45	S223	Headworks Modification Details
46	S271	Septage Receiving Station Plan and Details
47	S321	Primary Effluent Flow Control Structure Plan and Sections
48	S322	Primary Effluent Flow Control Structure Sections and Details
49	S421	Aeration Basin Bottom Plan
50	S422	Aeration Basin Partial Plan 1
51	S423	Aeration Basin Partial Plan 2
52	S424	Aeration Basin Sections and Details 1
53	S425	Aeration Basin Sections and Details 2
54	S426	Aeration Basin Sections 1
55	S427	Aeration Basin Sections 2
56	S431	Blower and Generator Building Floor and Foundation Plan
57	S432	Blower and Generator Building Roof Framing Plan
58	S433	Blower and Generator Building Sections
59	S434	Blower and Generator Building Sections and Details
60	S441	Secondary Clarifiers Bottom Plan
61	S442	Secondary Clarifiers Top Plan
62	S443	Secondary Clarifiers Sections
63	S444	Secondary Clarifier Details
64	S444	Secondary Clarifier and Pump Station Details
65	S601	Chlorine Contact Tank Upper Plan
67	S603	Chlorine Contact Tank Sections and Details 1
68	S604	Chlorine Contact Tank Sections and Details 2
69	S701	Solids Processing Building Plan
70	S702	Solids Processing Building Elevations and Details 1
71	S703	Solids Processing Building Elevations and Details 2
72	S704	Solids Processing Building Elevations and Details 3
73	S705	Recycle Pump Station Plans, Sections, and Details
74	S711	Anaerobic Digester Plan View
75	S712	Anaerobic Digester Section and Details 1
76	S713	Anaerobic Digester Section and Details 2
77	S714	Anaerobic Digester Section and Details 3
78	S715	Anaerobic Digester Details
79	S741	Sludge Storage and Pumping - Tank Modifications 1
80	S742	Sludge Storage and Pumping - Tank Modifications 2
81	A001	Miscellaneous Details 1
82	A002	Miscellaneous Details 2
83	A003	Door and Finish Schedule
84	A431	Blower and Generator Building Floor Plan
85	A432	Blower and Generator Building Exterior Elevations
86	A433	Blower and Generator Building Sections and Details 1
87	A434	Blower and Generator Building Sections and Details 2
88	A701	Solids Processing Building Floor Plan
89	A702	Solids Processing Building Exterior Elevations
90	A703	Solids Processing Building Sections and Details 1
91	A704	Solids Processing Building Sections and Details 2
92	M001	Mechanical Details 1
93	M002	Mechanical Details 2
94	M003	Mechanical Details 3
95	M004	Mechanical Details 4
96	M005	Mechanical Details 5
97	M006	Mechanical Details 6
98	M211	Influent Pump Station Plan
99	M212	Influent Pump Station Sections
100	M213	Influent Pump Station Sections and Details 1
101	M214	Influent Pump Station Sections and Details 2
102	M221	Headworks Modifications Plan
103	M222	Headworks Modifications Sections and Details
104	M271	Septage Receiving Station Plan
105	M272	Septage Receiving Station Sections and Details
106	M321	Primary Effluent Flow Control Structure Plans
107	M322	Primary Effluent Flow Control Structure Sections and Details
108	M421	Aeration Basin Plan
109	M422	Aeration Basin Sections 1
110	M423	Aeration Basin Sections 2
111	M424	Aeration Basin Sections and Details 1
112	M425	Aeration Basin Sections and Details 2
113	M431	Blower and Generator Building Plan
114	M432	Blower and Generator Building Sections
115	M433	Blower and Generator Building Sections and Details
116	M434	Blower and Generator Building HVAC, Plumbing, Notes, and Schedule
117	M441	Secondary Clarifiers Plan
118	M442	Secondary Clarifiers Sections 1
119	M443	Secondary Clarifiers Sections 2
120	M444	Secondary Clarifier and Pump Station Sections and Details
121	M601	Chlorine Contact Tank Upper Plan
122	M602	Chlorine Contact Tank Lower Plan
123	M603	Chlorine Contact Tank Sections
124	M604	Chlorine Contact Tank Sections and Details 1
125	M605	Chlorine Contact Tank Sections and Details 2
126	M701	Solids Processing Building General Plan
127	M702	Solids Processing Building Lower Plan
128	M703	Solids Processing Building HVAC and Plumbing Plan
129	M704	Solids Processing Building HVAC Air Flow Schematic
130	M705	Solids Processing Building Sections 1
131	M706	Solids Processing Building Sections 2
132	M707	Solids Processing Building Setions and Details
134	M711	Anaerobic Digester Lower Plan
135	M712	Anaerobic Digester Roof Plan
136	M713	Anaerobic Digester Partial Plan and Sections
137	M714	Anaerobic Digester Sections
138	M715	Anaerobic Digester Sections and Details 1
139	M716	Anaerobic Digester Sections and Details 2
140	M717	Waste Gas Burner Plan and Section
141	M741	Sludge Storage and Pumping - Modifications to Existing Secondary Clarifiers
142	M742	Sludge Storage and Pumping - Modifications to Existing South Primary Clarifier
143	M743	Sludge Storage and Pumping - Modifications to Existing Primary Digester
144	M744	Sludge storage and Pumping - Modifications to Existing Secondary Digester
145	M911	Front Street Chemical Dosing Station Plans
146	M912	Front Street Chemical Dosing Station Sections and Details
147	E001	Electical Abbreviations and Symbols
148	E002	Electrical Details 1
149	E003	Electrical Details 2
150	E004	Electrical Details 3
151	E004	Electrical Details 4
152	E011	Electrical Single Line Diagram
153	E21	Panelboard Schedules
154	E22	Conduit and Wire Schedules
155	E31	Elementary Diagrams
156	E32	Elementary Diagrams
157	E33	Elementary Diagrams
158	E41	Site Security Plan 1
159	E42	Site Security Plan 2
160	E43	Site Security Details
161	E101	Existing Treatment Facility Electrical Plan
162	E102	Electrical Site Plan (North)
163	E103	Electrical Site Plan (South)
164	E211	Influent Pump Station Power and Lighting Plan
165	E221	Headworks Power and Lighting Plan
166	E271	Septage Receiving Station Power and Lighting Plan
167	E321	Primary Effluent Flow Control Structure Power plan
168	E421	Aeration Basin Power and Lighting Plan
169	E431	Blower Building Power and Lighting Plan
170	E432	Blower Buidling Signal Plan
171	E433	Blower Building MCC Elevations
172	E441	Secondary Clarifiers Power and Lighting Plan
173	E701	Solids Processing Building Power Plan
174	E702	Solids Processing Building Lighting Plan
175	E703	Solids Processing Building Signal Plan
176	E704	Solids Processing Buiding MCC Elevations
177	E704	Recycle Pump Station Power Plan
178	E711	Anaerobic Digester Power Plan
179	E741	Sludge Storage and Pumping Power Plans
180	E911	Front Street Chemical Dosing Power and Lighting Plan
181	I001	Process and Instrumentation Symbols and Abbreviations
182	I002	PLC Panel Elevations and Details
183	I003	Miscellaneous Systems and SCADA
184	I004	Instrumentation Details
185	I221	Process and Instrumentation Influent Pump Station
186	I231	Process and Instrumentation Headworks
187	I271	Process and Instrumentation Septage Receiving Station
188	I331	Process and Instrumentation Primary Clarifier and Primary Effluent Flow Control Structure
189	I421	Process and Instrumentation Aeration Basins Liquid Stream
190	I422	Process and Instrumentation Aeration Basins Air
191	I431	Process and Instrumentation Aeration Air Blowers
192	1441	Process and Instrumentation Secondary Clarifier 1
193	I442	Process and Instrumentation Secondary Clarifier 2
194	I601	Process and Instrumentation Disinfection System 1
195	I602	Process and Instrumentation Disinfection System 2
196	I711	Process and Instrumentation Anaerobic Digester Sludge System
197	I712	Process and Instrumentation Anaerobic Digester Gas System
198	I713	Process and Instrumentation Anaerobic Digester Heat Reservoir System
199	I714	Process and Instrumentation Recycle Pump Station



Kennedy/Jenks Consultants

CITY OF TILLAMOOK

PROCESS FLOW SCHEMATIC

0491010.20

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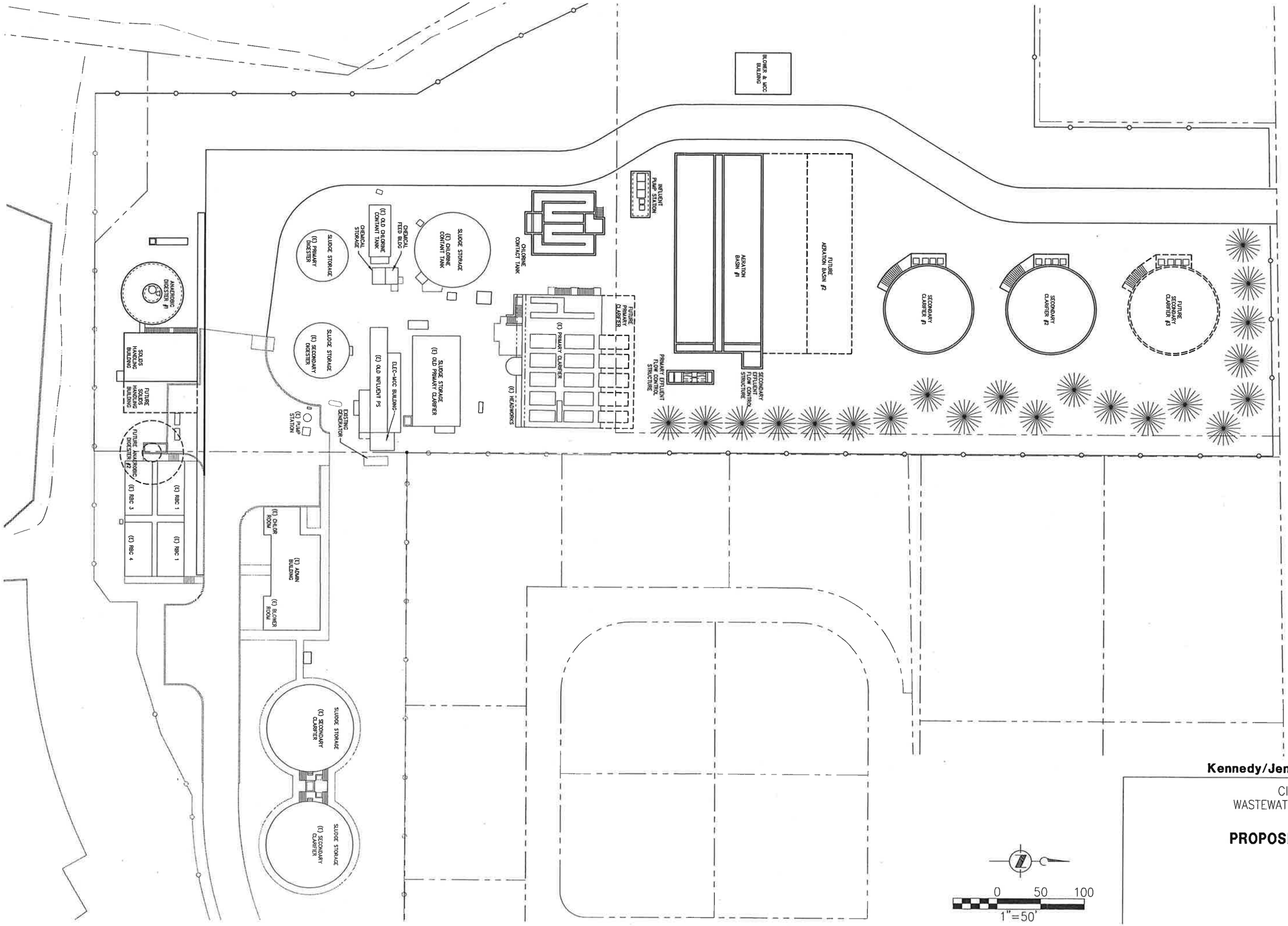
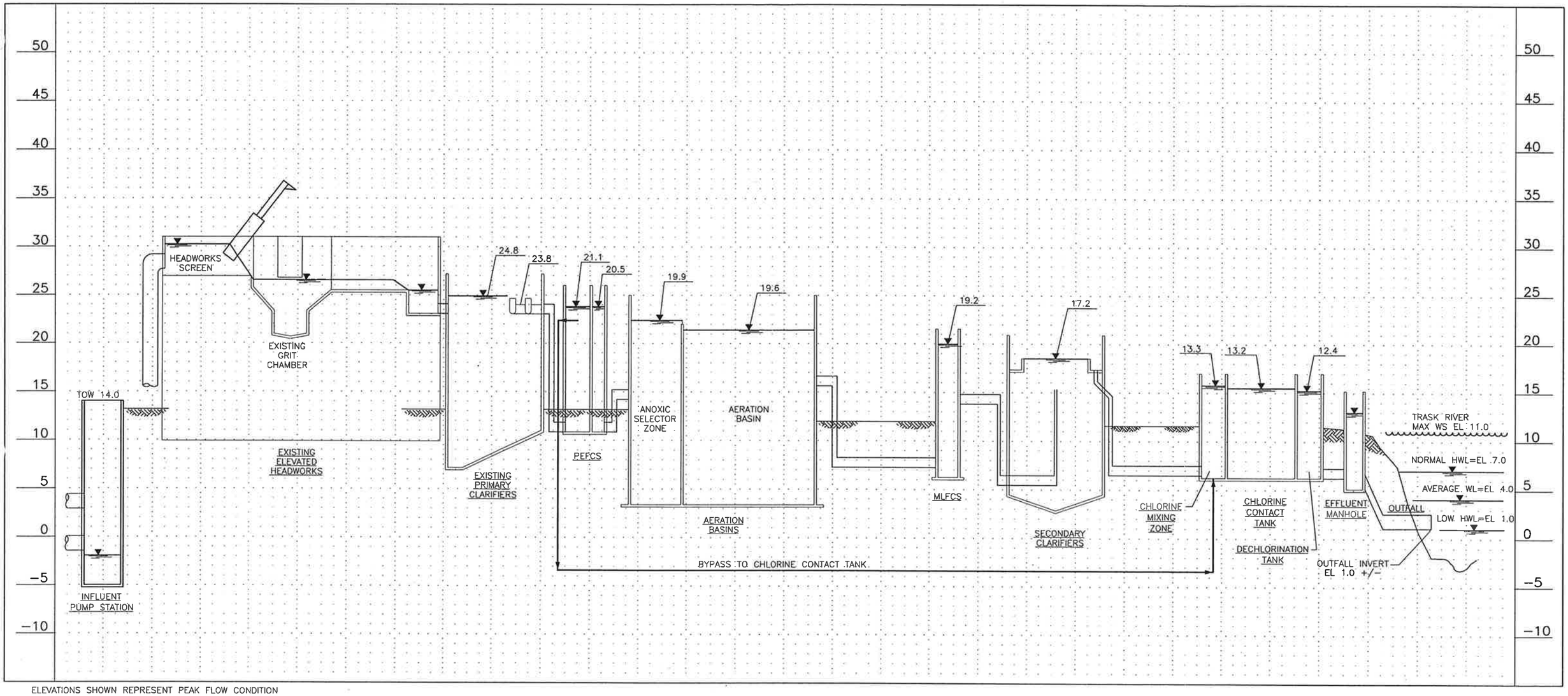


Kennedy/Jenks Consultants

CITY OF TILLAMOOK
WASTEWATER FACILITY PLAN

PROPOSED SITE PLAN

0491010.20



ELEVATIONS SHOWN REPRESENT PEAK FLOW CONDITION

Kennedy/Jenks Consultants

City of Tillamook

HYDRAULIC PROFILE

0491010.20

ENGINEER'S ESTIMATE OF PROBABLE COST
Project: City of Tillamook WWTP Expansion

KENNEDY/JENKS CONSULTANTS
Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No.: 491010.10
Months to Midpoint of Constr.: 18

SUMMARY BY AREA

ITEM NO.	ITEM DESCRIPTION		MATERIALS	INSTALLATION	TOTAL
1	Influent Pump Station		$277,000	$138,000	$415,000
2	Headworks Improvements		$77,000	$36,000	$113,000
3	Aeration Basin / Blowers / Air Piping		$531,000	$328,000	$859,000
4	Blower Building and Solids Processing Building		$209,000	$167,000	$376,000
5	Secondary Clarifiers and RAS / WAS / Scum Pumping		$542,000	$432,000	$974,000
6	Chlorine Contact Tank and Appurtenances		$133,000	$106,000	$239,000
7	WAS Thickener & Sludge Storage Improvements		$240,000	$61,000	$301,000
8	Digester and Appurtenances		$202,000	$116,000	$318,000
9	Septage Receiving Station		$94,000	$26,000	$120,000
10	Yard Piping and Outfall Improvements		$187,000	$144,000	$331,000
11	Hydraulic Control Structures		$43,000	$32,000	$75,000
12	Electrical / SCADA / Standby Power / Instrumentation		$619,000	$151,000	$770,000
13	Front Street Chemical Dosing Station		$64,000	$18,000	$82,000
14	Sitework, Landscaping, and Security		$113,000	$112,000	$225,000
	Subtotals		$3,331,000	$1,867,000	$5,198,000
	Construction Contingency @	25%	$833,000	$467,000	$1,300,000
	Contractor OH&P @	15%	$500,000	$280,000	$780,000
	Contractor Mobilization @	7%	$233,000	$131,000	$364,000
	Escalate to Midpt. of Const. @ (.5% per month)	9%	$300,000	$168,000	$468,000
	Total Estimated Construction Cost		**$5,197,000**	**$2,913,000**	**$8,110,000**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 1: Influent Pump Station

Prepared By: TDL
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10

Current at ENR _____
Escalated to ENR _____

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	Concrete - Walls	58	cuyd	$250	$14,500	$400	$23,200	$37,700
	2	Concrete - Slab on base	28	cuyd	$250	$7,000	$250	$7,000	$14,000
	3	Concrete - Slab on grade	11	cuyd	$250	$2,750	$250	$2,750	$5,500
	4	Drive sheeting	1,800	sf	$10	$18,000	$15	$27,000	$45,000
	5	Excavation	791	cuyd	$0	$0	$20	$15,820	$15,820
	6	Backfill	359	cuyd	$10	$3,590	$10	$3,590	$7,180
	7	Control of Water	1	ls	$0	$0	$10,000	$10,000	$10,000
	8	18 HP Pumps	2	ea	$12,000	$24,000	$2,400	$4,800	$28,800
	9	45 HP Pumps	3	ea	$28,500	$85,500	$5,700	$17,100	$102,600
	10	Custom access cover	1	ls	$6,000	$6,000	$1,200	$1,200	$7,200
	11	Custom Control Panel	1	ls	$44,000	$44,000	$8,800	$8,800	$52,800
	12	8-inch Plug Valve	3	ea	$850	$2,550	$170	$510	$3,060
	13	6-inch Plug Vlave	2	ea	$650	$1,300	$130	$260	$1,560
	14	8-inch Check valve	3	ea	$700	$2,100	$140	$420	$2,520
	15	6-inch Check Valve	2	ea	$600	$1,200	$120	$240	$1,440
	16	Discharge Manifold Fittings	2	ea	$15,000	$30,000	$3,000	$6,000	$36,000
	17	3X5 AL H20 HATCH	6	ea	$800	$4,800	$160	$960	$5,760
	18	ladder and railing	1	ls	$5,000	$5,000	$1,000	$1,000	$6,000
	19	Odor Control	1	ls	$10,000	$10,000	$5,000	$5,000	$15,000
	20	Bridge Crane & Hoist	1	ls	$15,000	$15,000	$2,000	$2,000	$17,000
Subtotals						**$277,000**		**$138,000**	**$415,000**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 2: Headworks Improvements

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Prepared By: BKH/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	Demo Parshall Flume & Grout	1	LS			$2,000	$2,000	$2,000
	2	Demo Comminutor & Controls	1	LS			$2,000	$2,000	$2,000
	3	Bypass Channel	1	LS	$5,000	$5,000	$12,500	$12,500	$17,500
	4	Handrailing	42	LF	$30	$1,260	$18	$756	$2,016
	5	Walkway Grating	65	SF	$15	$975	$5	$325	$1,300
	6	Fine Screen and Controls	1	EA	$60,000	$60,000	$13,000	$13,000	$73,000
	7	Manual Bar Screen	1	EA	$2,500	$2,500	$2,000	$2,000	$4,500
	8	Screenings Chute & Dumpster	1	EA	$2,500	$2,500	$500	$500	$3,000
	9	Jib Crane	1	EA	$1,500	$1,500	$250	$250	$1,750
	10	Staircase	1	EA	$2,500	$2,500	$1,000	$1,000	$3,500
	11	Handrailing Modifications (Removalble Sections)	1	LS	$1,000	$1,000	$1,500	$1,500	$2,500
Subtotals						**$77,000**		**$36,000**	**$113,000**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 3: Aeration Basin/Blowers/Air Piping

Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	Concrete - Walls	455	cuyd	$250	$113,750	$250	$113,750	$227,500
	2	Concrete - Slab on grade	385	cuyd	$250	$96,250	$250	$96,250	$192,500
	3	Misc Metals - grating	564	sf	$15	$8,460	$5	$2,820	$11,280
	4	Misc Metals - rails	320	lf	$30	$9,600	$18	$5,760	$15,360
	5	Misc Metals - stairs	12	ea	$300	$3,600	$0	$0	$3,600
	6	Partition Baffles	3	ea	$3,000	$9,000	$1,500	$4,500	$13,500
	7	Submersible Mixers	3	ea	$6,000	$18,000	$1,200	$3,600	$21,600
	8	Isolation Gates	8	ea	$4,000	$32,000	$1,000	$8,000	$40,000
	9	Weir Gates	3	ea	$4,000	$12,000	$1,000	$3,000	$15,000
	10	Recirc pump	1	ea	$7,500	$7,500	$1,500	$1,500	$9,000
	11	Aeration - Blowers	4	ea	$30,000	$120,000	$3,000	$12,000	$132,000
	12	Aeration - Diffuser	1	ls	$65,000	$65,000	$30,000	$30,000	$95,000
	13	8-inch SS Sch-10	250	lf	$65	$16,250	$5	$1,250	$17,500
	14	6-inch SS Sch-10	100	lf	$50	$5,000	$5	$500	$5,500
	15	4-inch SS Sch-10	150	lf	$52	$7,800	$5	$750	$8,550
	16	4-inch Butterfly valve	5	ea	$350	$1,750	$200	$1,000	$2,750
	17	Earthwork - Excavation and disposal	1,914	cuyd	$0	$0	$20	$38,280	$38,280
	19	Earthwork - Structural Fill	466	cuyd	$10	$4,660	$10	$4,660	$9,320
Subtotals						**531,000**		**328,000**	**858,000**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 4: Blower Building and Solids Processing Building

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	32' x 24' Blower Building	770	SF	$125	$96,250	$100	$77,000	$173,250
	2	30' X 30' Solids Processing Building	900	SF	$125	$112,500	$100	$90,000	$202,500
Subtotal						**$209,000**		**$167,000**	**$376,000**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 5: Secondary Clarifiers and RAS / WAS / Scum Pumping

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	Concrete - 12" walls	110	CY	$250	$27,500	$400	$44,000	$71,500
	2	Concrete - 12" slab	125	CY	$250	$31,250	$400	$50,000	$81,250
	3	Concrete - Launder	40	CY	$250	$10,000	$400	$16,000	$26,000
	4	Concrete - Pipe encasement	20	CY	$250	$5,000	$250	$5,000	$10,000
	5	Concrete - Sidewalk	16	CY	$250	$4,000	$250	$4,000	$8,000
	6	Earthwork - Excavation and Disposal	1,225	CY	$0	$0	$20	$24,500	$24,500
	7	Earthwork - Structural Fill	242	CY	$10	$2,420	$10	$2,420	$4,840
	8	Earthwork - Dewater	5	DAY	$178	$888	$385	$1,925	$2,813
	9	Clarifier Equipment	1	LS	$105,000	$105,000	$25,000	$25,000	$130,000
	10	18" DI	50	LF	$55	$2,750	$40	$2,000	$4,750
	11	16" DI	15	LF	$50	$750	$35	$525	$1,275
	12	12" DI	40	LF	$35	$1,400	$25	$1,000	$2,400
	13	6" DI	20	LF	$25	$500	$10	$200	$700
	14	16" Flexible Coupling	1	EA	$600	$600	$58	$58	$658
	15	6" Plug Valve	2	EA	$655	$1,310	$280	$560	$1,870
	16	12" Plug Valve	1	EA	$1,200	$1,200	$350	$350	$1,550
	17	18" 90 deg	1	EA	$1,120	$1,120	$325	$325	$1,445
	18	16" 90 deg	2	EA	$586	$1,172	$300	$600	$1,772
	19	12" 90 deg	1	EA	$225	$225	$275	$275	$500
	20	6" 90 deg	2	EA	$72	$144	$250	$500	$644
	21	Misc Metals - grating	55	sf	$15	$825	$5	$275	$1,100
	22	Misc Metals - rails	30	lf	$30	$900	$18	$540	$1,440
	23	Concrete - stairs	12	ea	$300	$3,600	$200	$2,400	$6,000
Subtotals for one clarifier						**$203,000**		**$182,000**	**$385,000**
							Total for two clarifiers		**$770,000**

Building, Area:		Item 5: RAS/WAS/Scum Pump Station							
Spec.	Item				Materials		Installation		
Section	No.	Description	Qty	Units	$/Unit	Total	$/Unit	Total	Total
	1	Concrete - wet well walls	12.0	cuyd	$250	$3,000	$400	$4,800	$7,800
	2	Concrete - wet well base	4.0	cuyd	$250	$1,000	$250	$1,000	$2,000
	3	Concrete - valve vault walls	7.0	cuyd	$250	$1,750	$400	$2,800	$4,550
	4	Concrete - valve vault base	4.0	cuyd	$250	$1,000	$250	$1,000	$2,000
	5	Hatch Covers	8.0	EA	$1,000	$8,000	$500	$4,000	$12,000
	6	Piping, Valves and Fittings	1	LS	$17,500	$17,500	$10,000	$10,000	$27,500
	7	RAS Pump (15 hp 1350 gpm)	1	LS	$15,000	$15,000	$2,500	$2,500	$17,500
	8	RAS pump (5 hp 450 gpm)	1	LS	$8,000	$8,000	$2,500	$2,500	$10,500
	9	WAS pump (2 hp 250 gpm)	1	LS	$5,000	$5,000	$2,500	$2,500	$7,500
	10	Scum Pump (5 hp 450 gpm)	1	LS	$8,000	$8,000	$2,500	$2,500	$10,500
	11	Davit Crane	1	EA	$1,500	$1,500	$250	$250	$1,750
	12	Pressure transducer	1	EA	$2,000	$2,000	$500	$500	$2,500
Subtotals for one clarifier						**$68,000**		**$34,000**	**$102,000**
							Total for two clarifiers		**$204,000**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 6: Chlorine Contact Tank and Appurtenances

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	Concrete - Walls	153	cuyd	$250	$38,250	$400	$61,200	$99,450
	2	Concrete - Slab on grade	54	cuyd	$250	$13,500	$250	$13,500	$27,000
	3	Earthwork - Excavation and Disposal	678	cy	$0	$0	$20	$13,560	$13,560
	4	Earthwork - Structural Fill	110	cy	$10	$1,100	$10	$1,100	$2,200
	5	Gates	4	ea	$4,000	$16,000	$1,000	$4,000	$20,000
	6	2 hp Flash Mixers and Controls	2	ea	$5,000	$10,000	$2,500	$5,000	$15,000
	7	Instrumentation and Chem. Feed.	1	ls	$20,000	$15,000	$5,000	$5,000	$20,000
	8	Misc Metals - grating	175	sf	$15	$2,625	$5	$875	$3,500
	9	Misc Metals - rails	100	lf	$30	$3,000	$18	$1,800	$4,800
	10	Misc Metals - stairs	12	ea	$300	$3,600	$0	$0	$3,600
	11	Chlorine handling equipment	1	ls	$30,000	$30,000	$0	$0	$30,000
Subtotals						**$133,000**		**$106,000**	**$239,110**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 7: WAS Thickener and Sludge Storage Improvements

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	Gravity Belt Thickener	1	EA	$130,000	$130,000	$10,000	$10,000	$140,000
	2	Pipe and Valves	4	EA	$1,000	$4,000	$100	$400	$4,400
	3	Polymer feed system	1	EA	$6,000	$6,000	$1,000	$1,000	$7,000
	4	Sludge Storage improvements	1	ls	$100,000	$100,000	$50,000	$50,000	$150,000
Subtotal						**$240,000**		**$61,000**	**$301,400**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 8: Digester and Appurtenances

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	Concrete - 12" walls	75	CY	$250	$18,750	$400	$30,000	$48,750
	2	Concrete - 24" slab	60	CY	$250	$15,000	$250	$15,000	$30,000
	3	Epoxy Coating	3,600	SF	$5	$18,000	$6	$21,600	$39,600
	4	Steel cover	1	EA	$50,000	$50,000	$10,000	$10,000	$60,000
	5	Earthwork - Excavation and Disposal	300	CY		$0	$20	$6,000	$6,000
	6	Earthwork - Structural Fill	120	CY	$10	$1,200	$10	$1,200	$2,400
	7	Boiler	1	EA	$9,000	$9,000		$0	$9,000
	8	Boiler Pumps (Hot water recirc)*	2	EA	$620	$1,240	$2,000	$4,000	$5,240
	9	Hot water recirc piping, etc.	1	EA	$10,000	$10,000	$3,000	$3,000	$13,000
	10	Expansion Tank	1	EA	$385	$385		$0	$385
	11	Air Separator	1	EA	$100	$100		$0	$100
	12	Heat Exchanger	1	EA	$27,500	$27,500	$5,000	$5,000	$32,500
	13	Mixing Pumps*	2	EA	$10,000	$20,000	$2,000	$4,000	$24,000
	13	Sludge pumps	2	EA	$7,000	$14,000	$2,000	$4,000	$18,000
	14	4" Plug Valve (sludge piping)	5	EA	$175	$875	$150	$750	$1,625
	15	4" DI (sludge piping)	100	LF	$10	$1,000	$10	$1,000	$2,000
	16	8" DI (suction & discharge)	300	LF	$25	$7,500	$20	$6,000	$13,500
	17	8" Plug Valve (suction & discharge)	5	EA	$1,100	$5,500	$350	$1,750	$7,250
	18	8" Tee	8	EA	$150	$1,200	$180	$1,440	$2,640
	19	8" 90 deg	9	EA	$120	$1,080	$115	$1,035	$2,115
Subtotal						**$202,000**		**$116,000**	**$318,105**

* Assumes one installed pump & one spare pump

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 9: Septage Receiving Station

Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	Equipment pad w/area drain	9	cuyd	$100	$900	$400	$3,600	$4,500
	2	Septage Receiving Station (w/ rock trap)	1	ea	$90,000	$90,000	$20,000	$20,000	$110,000
	3	Septage Storage Manhole	1	ls	$2,000	$2,000	$2,000	$2,000	$4,000
	4	Flexible Hose	1	ea	$100	$100	$100	$100	$200
	5	Hard Piping	1	ea	$1,000	$1,000	$500	$500	$1,500
Subtotal						**$94,000**		**$26,000**	**$120,200**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 10: Yard Piping and Outfall Improvements

Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
	1	24" DI	463	LF	$78.82	$36,454	$40	$18,500	$54,954
	2	20" DI	89	LF	$62.17	$5,533	$35	$3,115	$8,648
	3	18" DI	698	LF	$54.54	$38,042	$32	$22,320	$60,362
	4	16" DI	192	LF	$47.41	$9,103	$28	$5,376	$14,479
	5	12" DI	0	LF	$30.62	$0	$25	$0	$0
	6	8" DI	530	LF	$19.07	$10,107	$20	$10,600	$20,707
	7	6" DI	148	LF	$14.56	$2,155	$15	$2,220	$4,375
	8	4" DI	2,019	LF	$14.34	$28,945	$15	$30,278	$59,223
	9	24" PVC	20	LF	$45.00	$900	$40	$800	$1,700
	10	18" PVC	774	LF	$30.00	$23,228	$32	$24,776	$48,004
	10	24" 90 deg	6	EA	$1,273.71	$7,642	$325	$1,950	$9,592
	11	18" 90 deg	5	EA	$672.54	$3,363	$325	$1,625	$4,988
	12	16" 90 deg	2	EA	$372.15	$744	$300	$600	$1,344
	13	12" 90 deg	0	EA	$143.14	$0	$275	$0	$0
	13	10" 90 deg	3	EA	$104.83	$314	$250	$750	$1,064
	13	8" 90 deg	4	EA	$72.17	$289	$250	$1,000	$1,289
	14	6" 90 deg	2	EA	$46.37	$93	$250	$500	$593
	15	4" 90 deg	32	EA	$28.63	$916	$250	$8,000	$8,916
	16	24" Tee	2	EA	$1,744.65	$3,489	$250	$500	$3,989
	17	18" Tee	2	EA	$1,045.50	$2,091	$250	$500	$2,591
	18	16" Tee	2	EA	$524.16	$1,048	$250	$500	$1,548
	19	10" Tee	1	EA	$134.67	$135	$250	$250	$385
	20	8" Tee	3	EA	$96.36	$289	$250	$750	$1,039
	21	4" Tee	14	EA	$42.74	$598	$250	$3,500	$4,098
	22	Manholes	4	ea	$2,000	$8,000	$1,000	$4,000	$12,000
	23	outfall manhole	1	ea	$4,000	$4,000	$2,000	$2,000	$6,000
	24	Outfall RIP RAP	1	LS	$20,000	$20,000	$10,000	$10,000	$30,000
	25	Water Service improvements	1	LS	$30,000	$30,000	$10,000	$10,000	$40,000
Subtotals						**$187,000**		**$144,000**	**$331,888**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Prepared By: TDL/HER

Date Prepared: 15-Jan-06

Building, Area: Item 11: Hydraulic Control Structures

K/J Proj. No. 491010.10

Current at ENR

Escalated to ENR

Estimate Type:
- ☐ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
		PRIMARY EFF. FLOW CONTROL BOX							
	1	Concrete - Walls	26	cuyd	$250	$6,500	$400	$10,400	$16,900
	2	Concrete - Slab on grade	6	cuyd	$250	$1,500	$250	$1,500	$3,000
	3	Earthwork - Excavation and Disposal	10	cuyd	$0	$0	$20	$200	$200
	4	Earthwork - Structural Fill	5	cuyd	$10	$50	$10	$50	$100
	4	Misc Metals - grating	100	sf	$15	$1,500	$5	$500	$2,000
	4	Launder	24	lf	$90	$2,160	$10	$240	$2,400
	5	Weir	3	ea	$3,000	$9,000	$1,000	$3,000	$12,000
	6	Electric actuator	1	ea	$3,500	$3,500	$1,000	$1,000	$4,500
		SECONDARY CLAR. FLOW SPLITTER BOX							
	1	Concrete - Walls	12	cuyd	$150	$1,800	$500	$6,000	$7,800
	2	Concrete - Slab on grade	5	cuyd	$100	$500	$400	$2,000	$2,500
	3	Earthwork - Excavation and Disposal	10	cuyd	$0	$0	$20	$200	$200
	4	Earthwork - Structural Fill	5	cuyd	$10	$50	$10	$50	$100
	4	Misc Metals - grating	100	sf	$15	$1,500	$5	$500	$2,000
	5	Gates	3	ea	$2,000	$6,000	$1,000	$3,000	$9,000
	6	Flume	3	ea	$3,000	$9,000	$1,000	$3,000	$12,000
Subtotals						**$43,000**		**$32,000**	**$74,700**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 12: Electrical / SCADA / Standby Power / Instrumentation

Estimate Type: Conceptual
Preliminary (w/o plans)
Design Development @

Prepared By: PAR
Date Prepared: 30-Aug-05
K/J Proj. No. 0491010*10

Current at ENR _____
Escalated to ENR _____

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total
DIVISION 16 - ELECTRICAL SYSTEMS						$0		$0		$0	$0
16110		4" Conduit - PVC-40		LF	7.00	$0	7.40	$0		$0	$0
		4" Conduit - PVC Coated GRS		LF	30.80	$0	18.40	$0		$0	$0
		4" Conduit - GRS		LF	20.10	$0	10.00	$0		$0	$0
		3" Conduit - GRS	800	LF	13.60	$10,880	9.60	$7,680		$0	$18,560
		2½" Conduit - GRS		LF	10.90	$0	7.00	$0		$0	$0
		2" Conduit - GRS		LF	6.55	$0	6.00	$0		$0	$0
		1½ " Conduit - GRS	375	LF	4.90	$1,838	5.00	$1,875		$0	$3,713
		1" Conduit - PVC-40		LF	1.22	$0	2.65	$0		$0	$0
		1" Conduit - PVC Coated GRS		LF	5.26	$0	6.00	$0		$0	$0
		1" Conduit - GRS	450	LF	3.00	$1,350	2.75	$1,238		$0	$2,588
		¾" Conduit - PVC-40		LF	0.85	$0	2.15	$0		$0	$0
		¾" Conduit - PVC Coated GRS		LF	4.20	$0	4.75	$0		$0	$0
		¾" Conduit - GRS - power	6,240	LF	2.13	$13,291	2.75	$17,160		$0	$30,451
		¾" Conduit - GRS - control	9,300	LF	2.13	$19,809	2.75	$25,575			$45,384
16120		#14 AWG Conductor, control	41,400	LF	0.05	$2,070	0.25	$10,350		$0	$12,420
		#12 AWG Conductor, lighting	1,400	LF	0.07	$98	0.28	$392		$0	$490
		#12 AWG Conductor, power	21,240	LF	0.07	$1,487	0.28	$5,947			$7,434
		#10 AWG Conductor	2,120	LF	0.10	$212	0.32	$678		$0	$890
		#8 AWG Conductor		LF	0.15	$0	0.40	$0		$0	$0
		#6 AWG Conductor	1,800	LF	0.25	$450	0.55	$990		$0	$1,440
		#4 AWG Conductor	450	LF	0.38	$171	0.68	$306		$0	$477
		#2 AWG Conductor	1,500	LF	0.66	$990	0.61	$915		$0	$1,905
		#1 AWG Conductor		LF	0.76	$0	0.90	$0		$0	$0
		#2/0 AWG Conductor		LF	1.23	$0	0.95	$0		$0	$0
		#3/0 AWG Conductor		LF	1.51	$0	1.10	$0		$0	$0
		#250 KCMIL Conductor		LF	2.28	$0	1.37	$0		$0	$0
		#350 KCMIL Conductor	3,200	LF	3.15	$10,080	1.52	$4,864		$0	$14,944
		#500 KCMIL Conductor		LF	5.00	$0	1.92	$0		$0	$0
16124		#18 TSP Signal Cable	2,400	LF	0.40	$960	0.44	$1,056		$0	$2,016

Section	No.	Description	Qty	Units	$/Unit	Total	$/Unit	Total	$/Unit	Total	Total
		12-pair fiber cable		LF		$0		$0			$0
		6-pair fiber cable		LF	1.32	$0		$0			$0
16130		Junction Box		EA	10.10	$0	35.50	$0		$0	$0
		Junction Box, Weatherproof		EA	20.00	$0	35.50	$0		$0	$0
		Pullbox		EA	20.00	$0	35.50	$0		$0	$0
16140		Switch		EA	10.00	$0	15.00	$0		$0	$0
		Switch, 3-way	4	EA	25.00	$100	15.00	$60		$0	$160
		Receptacles		EA	10.00	$0	15.00	$0		$0	$0
		Receptacles, GFI		EA	20.00	$0	15.00	$0		$0	$0
		Receptacles, Weatherproof, GFI		EA	35.00	$0	15.00	$0		$0	$0
16155		Solid State Starter, 200 hp, 480V, 3PH		EA	10,000.00	$0	1,500.00	$0		$0	$0
		FVNR Starter, Size 1, 10 HP max	30	EA	680.00	$20,400	175.00	$5,250			$25,650
		FVNR Starter, Size 2, 25 HP max	1	EA	1,000.00	$1,000	175.00	$175			$1,175
		FVNR Starter, Size 3, 50 HP max		EA	1,400.00	$0	175.00	$0			$0
		FVNR Starter, Size 4, 100 HP max	3	EA	3,000.00	$9,000	175.00	$525		$0	$9,525
		FVNR Starter, Size 5, 200 HP max		EA		$0	175.00	$0			$0
		AFD, Size 1	1	EA	5,000.00	$5,000		$0			$5,000
		AFD, Size 2	2	EA	12,000.00	$24,000		$0			$24,000
		AFD, Size 3	3	EA	27,000.00	$81,000		$0			$81,000
		AFD, Size 4		EA	45,000.00	$0		$0			$0
		AFD, Size 5		EA	70,000.00	$0		$0			$0
16160		Distribution Switchboard, 480V, 3PH	1	EA	2,350.00	$2,350	1,000.00	$1,000		$0	$3,350
		Lighting Panelboard, 120/208V, 3PH, 42 cct	1	EA	4,400.00	$4,400	1,900.00	$1,900		$0	$6,300
16180		Service Disconnect	1	EA	2,500.00	$2,500	1,000.00	$1,000		$0	$3,500
		Meter	1	EA	800.00	$800	500.00	$500		$0	$1,300
		Current Transformer	1	EA	1,800.00	$1,800	200.00	$200		$0	$2,000
		Motor Disconnect Switch, 30A, NEMA 1	31	EA	100.00	$3,100	100.00	$3,100			$6,200
		Motor Disconnect Switch, 60A, NEMA 1	1	EA	120.00	$120	100.00	$100			$220
		Motor Disconnect Switch, 100A, NEMA 1	3	EA	200.00	$600	100.00	$300			$900
		Motor Disconnect Switch, 200A, NEMA 1		EA	450.00	$0	200.00	$0			$0
		Motor Disconnect Switch, 400A, NEMA 1		EA		$0	200.00	$0			$0
		Motor Disconnect Switch, other, NEMA 1		EA		$0	200.00	$0		$0	$0
		Motor Disconnect Switch, 30A, NEMA 4X		EA	1,500.00	$0	100.00	$0			$0
		Motor Disconnect Switch, 60A, NEMA 4X	2	EA	1,800.00	$3,600	100.00	$200			$3,800
		Motor Disconnect Switch, 100A, NEMA 4X	3	EA	3,600.00	$10,800	100.00	$300			$11,100
		Motor Disconnect Switch, 200A, NEMA 4X		EA	5,000.00	$0	200.00	$0			$0
		Motor Disconnect Switch, 400A, NEMA 4X		EA	10,000.00	$0	200.00	$0			$0
		Motor Disconnect Switch, other, NEMA 4X		EA		$0	200.00	$0			$0
16205		Standby Generator Set, 20kW		EA	14,000.00	$0	5,500.00	$0			$0
		Standby Generator Set, 60kW		EA		$0	5,500.00	$0			$0
		Standby Generator Set, 100kW		EA		$0	5,500.00	$0			$0

Section	No.	Description	Qty	Units	$/Unit	Total	$/Unit	Total	$/Unit	Total	Total
		Standby Generator Set, 200kW		EA	45,000.00	$0	5,500.00	$0			$0
		Standby Generator Set, 400kW	1	EA	100,000.00	$100,000	5,500.00	$5,500			$105,500
		Standby Generator Set, 500kW	0	EA		$0	5,500.00	$0		$0	$0
16250		Automatic Transfer Switch, 400A		EA	10,000.00	$0	650.00	$0		$0	$0
		Automatic Transfer Switch, 800A	1	EA	15,000.00	$15,000	650.00	$650			$15,650
		Automatic Transfer Switch, 1200A		EA	20,000.00	$0	650.00	$0			$0
16402		Underground Electrical	1	LS	2,000.00	$2,000	1,500.00	$1,500		$0	$3,500
		Concrete Encasement		LS	1,500.00	$0	1,000.00	$0		$0	$0
		Transformer Pad		EA	2,000.00	$0	750.00	$0		$0	$0
		Handholes	5	EA	300.00	$1,500	400.00	$2,000		$0	$3,500
16450		Grounding	1	LS	2,000.00	$2,000	500.00	$500		$0	$2,500
16460		Dry Type Transformer, 480-240/120, 3PH, 100 kVA	1	EA	10,000.00	$10,000	670.00	$670		$0	$10,670
		Dry Type Transformer, 480-120/208, 3PH, 45 kVA	1	EA	5,000.00	$5,000	670.00	$670			$5,670
16500		Fixture, T8 Fluorescent, Interior	24	EA	81.00	$1,944	45.00	$1,080		$0	$3,024
		Fixture, Metal Hallide, Bay Lighting		EA	255.00	$0	80.00	$0		$0	$0
		Fixture, Exterior	4	EA	120.00	$480	45.00	$180			$660
		Fixture, Yard, pole mounted	3	EA	1,000.00	$3,000	500.00	$1,500			$4,500
16955		Control Station		EA	172.00	$0	71.00	$0		$0	$0
16010		Misc Electrical	1	LS	1,000.00	$1,000	500.00	$500		$0	$1,500
		Generator Disconnect Panel		LS	10,000.00	$0	6,000.00	$0			$0
		Startup	1	LS		$0		$0	5,000.00	$5,000	$5,000
		Training	1	LS		$0		$0	0.00	$0	$0
						$0		$0			$0
						$0		$0			$0
						$0		$0			$0
SUBTOTAL - DIVISION 16						**$376,180**		**$108,386**		**$5,000**	**$489,566**
DIVISION 17 - INSTRUMENTATION						$0		$0		$0	$0
17110		Chlorine Analyzer	1	EA	2,500.00	$2,500	500.00	$500		$0	$3,000
		TSS transmitter and probe		EA	3,500.00						$0
		Turbidity transmitter and probe		EA	3,500.00						$0
		Analyzer - combined Ammonia/phosphate/nitrate		EA	41,300.00						$0
		DO Transmitter and probe	2	EA	3,100.00	$6,200	500.00	$1,000		$0	$7,200
		pH Analyzer and probe	2	EA	2,000.00	$4,000	500.00	$1,000		$0	$5,000
17120		Flowmeter, 1" Magnetic		EA	2,500.00	$0	500.00	$0			$0
		Flowmeter, 2" Magnetic	2	EA	3,500.00	$7,000	500.00	$1,000		$0	$8,000
		Flowmeter, 3" Magnetic		EA	2,100.00	$0	500.00	$0		$0	$0
		Flowmeter, 4" Magnetic		EA	2,150.00	$0	500.00	$0		$0	$0
		Flowmeter, 6" Magnetic	3	EA	2,500.00	$7,500	500.00	$1,500		$0	$9,000

Section	No.	Description	Qty	Units	$/Unit	Total	$/Unit	Total	$/Unit	Total	Total
		Flowmeter, 8" Magnetic		EA	2,700.00	$0	500.00	$0		$0	$0
		Flowmeter, 12" Magnetic		EA	4,500.00	$0	500.00	$0		$0	$0
		Flowmeter, 16" Magnetic		EA	7,900.00	$0	500.00	$0		$0	$0
		Flowmeter, thermal dispersion	1	EA	600.00	$600					$600
17140		Submersible pressure transmitter		EA	2,000.00	$0	500.00	$0		$0	$0
		Level transmitter - ultrasonic		EA	1,200.00		500.00				$0
		Level transmitter - bubbler system		EA	4,000.00		1,000.00				$0
		Level transmitter - resistive	1	EA	2,000.00	$2,000	500.00	$500		$0	$2,500
		Float switch		EA	150.00						$0
17150		Pressure transmitter	1	EA	800.00	$800	500.00	$500		$0	$1,300
		Pressure switch		EA	300.00		500.00				$0
		Pressure gauge		EA	200.00		500.00				$0
		Pressure seals		EA	1,500.00	$0	500.00	$0		$0	$0
Miscellaneous		Proximity / Intrusion Switch		EA	200.00		200.00				$0
		Temperature transmitter		EA	800.00		500.00				$0
		Temperature gauge		EA	200.00		500.00				$0
		Smoke detector		EA							$0
		Manual Pull Station		EA							$0
		Security Alarm System and Control Panel		EA							$0
		Fire Alarm System and Control Panel	1	EA	7,000.00	$7,000	3,000.00	$3,000			$10,000
17330		PLC System (see separate sheet)	1	LS	20,000.00	$20,000		$0		$0	$20,000
		PLC Programming (plus software licenses)	1	LS	100,000.00	$100,000		$0		$0	$100,000
		SCADA System Turnkey	1	LS	80,000.00	$80,000	1,000.00	$1,000			$81,000
17425		Radio		EA	2,000.00	$0	1,000.00	$0		$0	$0
		Antenna		EA	1,000.00	$0	500.00	$0		$0	$0
17510		Main Control Panel	1	EA	5,000.00	$5,000	1,000.00	$1,000		$0	$6,000
17010		Instrument Calibration and Testing	1	LS		$0		$0	1,500.00	$1,500	$1,500
		Startup	1	LS		$0		$0	5,000.00	$5,000	$5,000
		Training	1	LS		$0		$0	20,000.00	$20,000	$20,000
				LS		$0		$0		$0	$0
											$0
SUBTOTAL - DIVISION 17						**$242,600**		**$11,000**		**$26,500**	**$280,100**
TOTALS						**$619,000**		**$119,000**		**$32,000**	**$769,666**

ENGINEER'S ESTIMATE OF PROBABLE COST

Project: City of Tillamook WWTP Expansion

Building, Area: Item 13: Front Street Chemical Dosing Station

KENNEDY/JENKS CONSULTANTS

Prepared By: TDL/HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10

Estimate Type:

- ☑ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development

Item Description	Quantity	Unit	Materials	Installation	Total
500 Gal XHDPE Storage Tank w/ Liner	1	ea	$3,500	$1,000	$4,500
NaOCl Metering Pumps	1	ea	$1,000	$1,250	$2,250
Piping, Valves,sight glass and Acc.	1	LS	$5,000	$2,000	$7,000
Fiberglass Shelter on concrete slab	1	LS	$30,000	$1,000	$31,000
Vault	1	LS	$3,000	$1,000	$4,000
3-inch Flowmeter	1	LS	$2,100	$1,000	$3,100
Forcemain Piping	1	LS	$6,000	$4,000	$10,000
Electrical/Instrumentation	1	LS	$10,000	$5,000	$15,000
Site Fencing	1	LS	$3,000	$2,000	$5,000
Subtotal			**$64,000**	**$18,000**	**$81,850**

ENGINEER'S ESTIMATE OF PROBABLE COST

KENNEDY/JENKS CONSULTANTS

Project: City of Tillamook WWTP Expansion

Building, Area: Item 14: Sitework, Landscaping, and Security

Prepared By: HER
Date Prepared: 15-Jan-06
K/J Proj. No. 491010.10
Current at ENR
Escalated to ENR

Estimate Type:
- ☑ Conceptual
- ☑ Preliminary (w/o plans)
- ☐ Design Development @
- ☐ Construction
- ☐ Change Order
- % Complete

Spec. Section	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Total
		Civil							
	1	Perimeter Fence and Gates	1,300	LF	$30	$39,000	$10.00	$13,000	$52,000
	2	Asphalt Paving	16,800	SF	$2.00	$33,600	$1.50	$25,200	$58,800
	3	Site Drainage	1	LS	$10,000	$10,000	$5,000.00	$5,000	$15,000
	4	Irrigation System	1	LS	$7,500	$7,500	$3,500	$3,500	$11,000
	5	Landscaping	1	LS	$10,000	$10,000	$5,000	$7,500	$17,500
		Subtotal Civil				**$100,100**		**$54,200**	**$154,300**
		Demolition							
	6	RBCs	50,000	cuft	$0	$0	$0.50	$25,000	$25,000
	7	Existing Pump Building	50,000	cuft	$0	$0	$0.50	$25,000	$25,000
		Subtotal Demolition				**$0**		**$50,000**	**$50,000**
		Security System							
	8	Surveillance Camera and Mount	2	EA		$7,500	$2,000	$4,000	$11,500
	9	Security Panel	1	EA		$5,000	$2,000	$2,000	$7,000
	10	Door and Window Contacts	18	EA		$15	$125	$2,250	$2,265
		Subtotal Security System				**$12,515**		**$8,250**	**$20,765**
	Subtotal					**$113,000**		**$112,000**	**$225,065**

Technical Memorandums

1. Revised Influent Loading Criteria
2. Influent Pump Station
3. Headworks Improvements
4. Liquid Flow Metering
5. Flexible Aeration Basin, Blowers, and Air Piping
6. Secondary Clarification
7. RAS/WAS/Scum Pumping
8. Chlorine Contact Tank and Disinfection Improvements
9. Yard Piping, Hydraulic Control Structures and Plant Utilities
10. Outfall Improvements and COE Permitting
11. Waste Activated Sludge Thickening
12. Anaerobic Digestion and Sludge Storage
13. Solids Processing Building/Blower and Standby Generator Building
14. Septage Receiving Station
15. Electrical, Instrumentation, and SCADA
16. Civil Work, Landscaping and Site Security
17. Front Street Sodium Hypochlorite Dosing Station

Technical Memorandum No. 1

Subject: Revised Influent Loading Criteria
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted by: Harry Ritter, P.E.

Introduction

In the spring of 2003, a Corrective Action Plan (CAP) was completed at the WWTP. The CAP incorporated four significant improvements for record keeping and influent sampling including:

- Installation of a new effluent flow meter and recalibration of the existing influent flow meter.
- Construction of a new digester supernatant pump station, allowing the digester recycle stream to return downstream of the influent sampler.
- Electrical improvements to provide the influent flow meter signal to the influent sampler so that samples could be taken proportional to flow.
- Relocation of the influent sampler upstream of the secondary solids discharge point in the headworks channel (secondary solids are co-thickened with primary sludge).

As part of the initial Wastewater Facility Plan effort, influent BOD and TSS data from January 1997 to early 2003 were examined but rejected as the basis for future design criteria due to errors resulting from the deficiencies noted above. As an alternative to using the actual operating records, the Facility Plan influent load projections were based on a simplified approach of applying an average load factor to the population served. The BOD and TSS loads were assumed to be constant year-round regardless of dry and wet weather flow patterns or seasonal variations in loading patterns. This was a reasonable assumption since Tillamook has a consistent year-round population, is not significantly affected by seasonal tourism, and does not normally experience significant periodic or seasonal variations in sewage strength from its existing commercial customers.

To check the validity of the loading estimates made in the Facility Plan, operating data for the period following completion of the CAP improvements were examined. The results of this analysis are detailed in this memorandum.

Flow and Loading Data

Flow and load data from the May 2003 through September 2005 Daily Monitoring Reports (DMRs) were summarized and graphed. The data appears in Table 1.1 and is shown graphically on Figures 1.1, 1.2 and 1.3. BOD and TSS are typically sampled 9-12 days each month for two or three consecutive days at a time.

Table 1.1 – Flow and Loading Data (May 2003 to Sept 2005)

Month	Monthly Average Flow (MGD)	Monthly Average BOD (mg/l)	Monthly Average BOD (ppd)	Monthly Average TSS (mg/l)	Monthly Average TSS (ppd)
May-03	0.96	142	1,017	113	1,058
Jun-03	0.58	211	992	251	1,156
Jul-03	0.41	316	1,023	435	1,427
Aug-03	0.40	304	1,014	357	1,178
Sep-03	0.40	299	998	307	1,014
Oct-03	0.74	187	1,238	177	1,096
Nov-03	1.18	158	1,311	187	1,691
Dec-03	1.80	115	1,625	118	1,662
Jan-04	1.78	91	1,079	124	1,670
Feb-04	1.54	78	958	79	978
Mar-04	1.06	143	1,225	141	1,143
Apr-04	0.86	158	1,172	170	1,266
May-04	0.72	268	1,567	268	1,618
Jun-04	0.86	164	1,078	159	1,083
Jul-04	0.49	326	1,310	263	1,064
Aug-04	0.57	294	1,370	314	1,245
Sep-04	0.87	219	1,240	208	1,187
Oct-04	1.02	155	1,354	173	1,454
Nov-04	0.97	149	1,209	149	1,211
Dec-04	1.32	124	1,189	110	1,082
Jan-05	1.16	148	1,350	184	1,710
Feb-05	0.78	192	1,203	158	978
Mar-05	0.89	211	1,301	203	1,307
Apr-05	1.30	123	1,176	150	1,595
May-05	0.90	172	1,331	182	1,372
Jun-05	0.88	186	1,295	174	1,161
Jul-05	0.58	230	1,112	237	1,144
Aug-05	0.474	274	1,059	321	1,193
Sep-05	0.437	425	1,417	631	2,157
Avg. of Monthly Averages		**202**	**1,214**	**219**	**1,307**

Figure 1.1 – Monthly Average Influent Flow

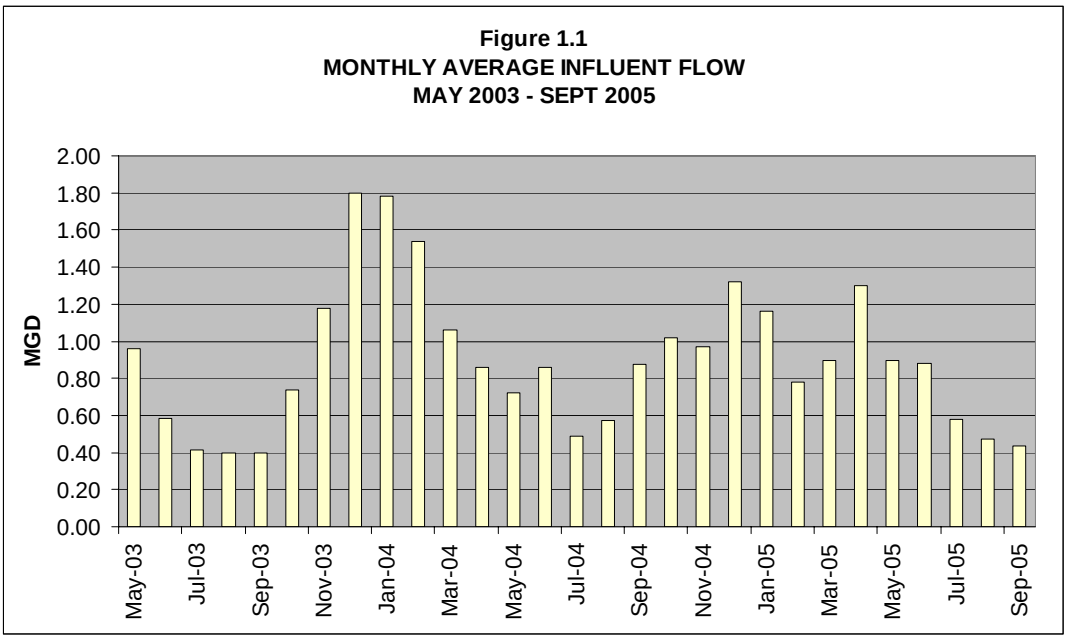


Figure 1.2 – Monthly Average Influent BOD & TSS Concentration

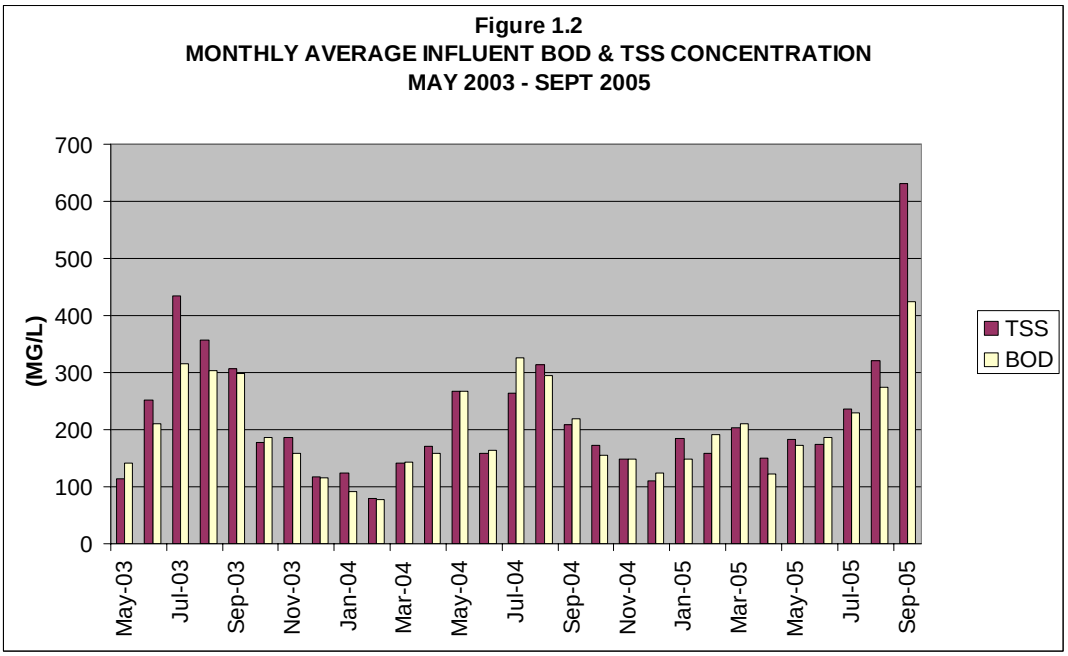
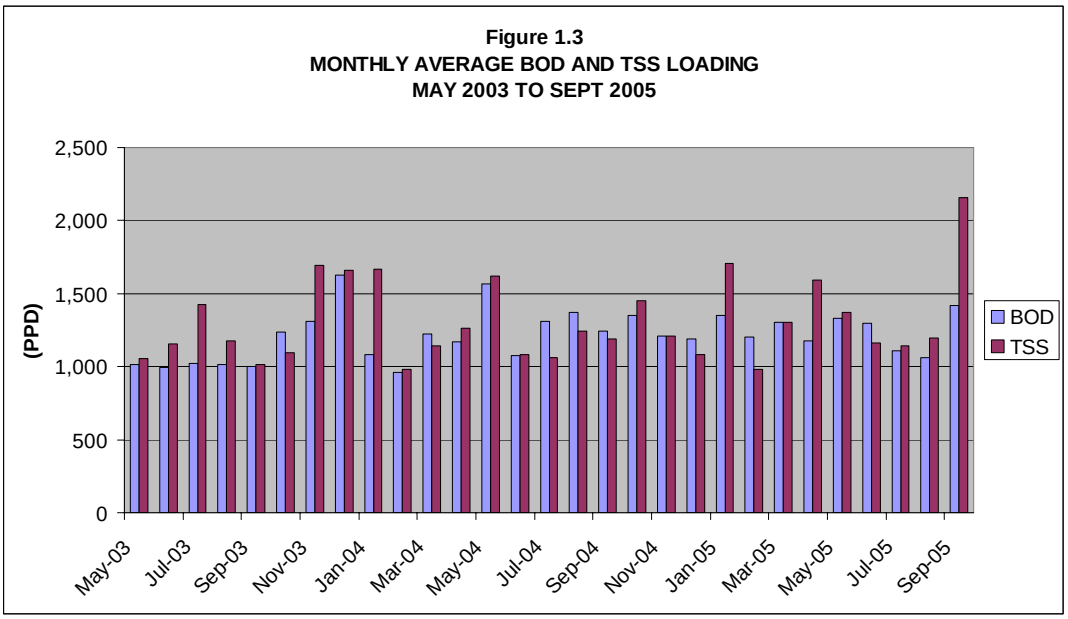


Figure 1.3 – Monthly Average BOD and TSS Loading



Discussion

The DMR data indicate that the BOD and TSS concentrations are, in general, inversely proportional to influent flow. The data also indicate that the monthly average BOD loading to the plant is in the range of 1,000 ppd to 1,350 ppd with peak monthly averages up to 1,625 ppd. The monthly average and monthly peak TSS concentration falls within similar ranges. There is no consistent seasonal pattern to the loading.

In the absence of a seasonal loading pattern, it is concluded that treatment plant operation and sampling likely skew the data. This is documented by a wide range of daily loadings recorded for months with nearly consistent daily flow values. The data taken as a whole, however, indicates a fairly consistent trend toward the average value. Based on discussions with the treatment plant staff, the most likely causes for wide variations in the daily loading data under uniform flow conditions are:

- The affect of rags, paper, and plastic material fouling the influent sampling tube
- Design and location of sampler intake
- Periodic recycling of basin washdown water back to the influent pump station

- Periodic, inadvertent recycling of digester solids and digester supernatant back into the influent pump station
- Periodic dumping of STEP tank contents into the treatment plant.

Rags, paper, and plastic products fouling the influent sampler is a common occurrence as there is no fine screen to remove this material. When this occurs, the sample may not be representative since these materials tend to filter out and dilute the strength of the sample. Installation of the proposed fine screen should remedy this problem.

The sampler intake design and location are not optimal for collecting samples over a wide range of flow conditions. During low flow, the sampler can draw in solids that have been deposited along the bottom of the headworks channel. This is, in part, due to the low velocity and shallow depth present in the channel during periods of low flow. The design and location of the sampler intake will be studied and optimized during the plant expansion project.

During routine basin cleaning, solids wasting and transfer of stored solids, it is possible for washwater, stored solids, and supernatant to be recycled back into the influent pump station. When this happens during a sampling period, high values for BOD and TSS can be obtained. Improved ability to manage the solids stream in the upgraded treatment plant should reduce or eliminate the occurrence of these unintentional recycle streams.

STEP tank disposal to the treatment plant is a potential source of high strength BOD and ammonia. If disposal of a STEP tank occurs on a test day, it will tend to skew the data to the high side. Conversely, STEP tank disposal occurring when the sampler is inactive could result in under reported BOD loads. Since both scenarios are equally likely to occur, it is assumed that the monthly average value is statistical representative value. STEP tank testing done in the summer of 2005 found a range of BOD, TSS, and ammonia from 150 to 1250 mg/l, 180 to 330 mg/l, and 32 to 52 mg/l respectively. If an average concentration of 650 mg/l BOD, 250 mg/l TSS, and 42 mg/l ammonia is assumed for STEP system septage, this equates to annual loadings of 705 lbs, 270 lbs, and 45 lbs respectively. This is roughly equal to one day's average BOD and ammonia loading to the WWTP. While STEP tank disposal may impact daily loading values from time to time, it is not a significant factor in the overall annual loading to the treatment plant.

Periodic loadings of high strength influent are rare but do occur. On two occasions in 2004, a local dairy farmer dumped spoiled milk into the collection system. Other anomalies in sewage strength and characteristics have occurred over the years.

Except for December 2003 and May 2004, peak month loadings normally fall around 1375 ppd for BOD and TSS. Sampling abnormalities may have contributed to elevated loadings recorded for those two months.

Conclusion

Analysis of the DMR records from the past two years indicates that the Facility Plan likely understated the daily BOD and TSS load estimate. Additionally, while the daily BOD and TSS load is independent of the season, periodic changes in plant operations likely account for the approximate 10% difference between the average and peak monthly values.

Based on the DMR information collected over the past two years, the monthly average BOD and TSS contribution including all residential and commercial sources is approximately 1,250 ppd. The average peak BOD and TSS is approximately 1,500 ppd. Correlated to a current population of approximately 4,500, this equates to a per capita BOD and TSS contribution of approximately 0.28 ppd on an annual average basis and 0.33 ppd for the peak month.

Recommendations

Based on the DMR data presented herein and the discussion presented above, it is recommended that the BOD and TSS design loading for the WWTP expansion reflect the criteria presented in Table 1.2. These values are based on the current population of 4,500 and the 20-year design population of 7,500. Ammonia criteria will be based on a contribution of 0.03 ppd per person which is within the range commonly encountered in domestic sewage.

Table 1.2 – Design Criteria for Influent BOD, TSS, and Ammonia

Criteria	Unit	Startup	Design
Average Month BOD	ppd	1,250	2,100
Peak Month BOD	ppd	1,485	2,475
Average Month TSS	ppd	1,250	2,100
Peak Month TSS	ppd	1,485	2,475
Ammonia (Average Month)	ppd	135	225
Ammonia (Peak Month)	ppd	270	450

Technical Memorandum No. 2

Subject: Influent Pump Station
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Harry Ritter, P.E.

Introduction

The existing influent pump station is part of the original WWTP and is over 50-years old. It has insufficient capacity to meet the current peak flows and the wetwell is too shallow, resulting in a hydraulic restriction of one of the two major conveyance lines serving the plant. Therefore, a new influent pump station will be constructed as part of the upcoming WWTP expansion.

Wastewater from the City's wastewater service area is conveyed to the WWTP from two, 18-inch gravity pipes. One pipe enters the existing influent pump station from the north and conveys flow from the oldest parts of the collection system. The other pipe cuts diagonally across the existing WWTP site from southeast to northwest and convey flow from the 12th Street Pump Station and the eastern portions of the collection system. Both pipes are concrete. The invert of the pipe from the north is approximately 12-feet below grade at the pump station. The invert of the pipe from the southeast is approximately 16-feet below grade virtually matching the floor elevation of the pump station wetwell.

The existing pump station has two original wetwell/drywell pumps and one submersible pump. The submersible pump was retrofit to the pump station in 1978 to increase peak capacity. The current peak capacity of the station (with all pumps pumping) is approximately 4.5 MGD at normal wetwell levels and approximately 5.0 MGD at elevated wetwell levels. Operating records indicate the pump station occasionally runs at 100% capacity under heavy rainfall conditions.

The City's collection system is old and experiences significant I/I during the wet weather months. Several manholes have a history of overflowing. Since the system has overflowing manholes and the existing pump station has a limited capacity, the ultimate peak day and peak instantaneous flow is unknown. The Facility Plan, using the statistical method developed by the DEQ for estimating peak wet weather flows, predicts a peak day flow of 7.2 MGD and a peak instantaneous flow of 10.0 MGD. The methodology used to establish this flow prediction has been reviewed and approved by the DEQ. The new pump station will be sized to meet this capacity.

New Pump Station Location

The proposed pump station location, shown on Figure I-2, the WWTP Expansion Site Plan, meets these requirements:

- it can be accessed by both existing gravity lines serving the WWTP
- it allows for efficient routing of forcemains from the pump station to the primary clarifier

- it allows for efficient routing of electrical conduit from the MCC in the new Blower Building to the new pump station
- it can be easily accessed by a service vehicle

Wetwell Depth

The wetwell of the new pump station will be approximately 22-feet deep depending on the specific pump submergence requirements.

Design Alternatives

Three types of stations were considered including a drywell/wetwell configuration, a submersible pump configuration, and a vertical turbine solids handling (VTSH) configuration. All three types of stations offer comparable performance and similar efficiencies. While the drywell/wetwell design offers some advantages from a maintenance viewpoint, it is the most expensive of the three alternatives by a significant margin and was rejected for that reason. VTSH pump motors are readily accessible for ease of maintenance but access to the pumps would require the use of a boom truck to extract the long column from the deep wetwell. In addition, these pumps have a limited selection in lower flow ranges and are typically more expensive than a submersible pump for a given flow range. Submersible pumps are available in a wide range of flows, are the least expensive of the three types of pumps considered and can be easily removed and replaced for maintenance purposes.

Recommended Alternative

A variable speed pump station using non-clog submersible pumps is the recommended alternative based on capital cost and ease of maintenance. In order to meet EPA reliability requirements, the pump station must be able to meet the peak instantaneous flow requirement with the largest pump out of service. The pump station must also be able to cover the wide range of flows predicted for the treatment plant.

Design Flows

The design flows for the proposed pump station are shown in Table 2.1.

Table 2.1 – Design Flows

Criteria	Flow (MGD)
Minimum Flow	0.25
Average Dry Weather Flow	0.9
Average Wet Weather Flow	2.5

Criteria	Flow (MGD)
Max. Month Average Flow	4.0
Peak Day Flow	7.2
Peak Instantaneous Flow	10.0

Wetwell, Valving, and Forcemain

The pump station will use a trench style wetwell. In this type of wetwell, the pumps are aligned along a row within a narrow trench. Influent enters the wetwell at one end of the trench. An Ogee ramp, located at the inlet end of the wetwell, provides a smooth transition for the influent to prevent the entrapment of air and to minimize splashing and the release of odors. The ramp also provides for periodic scouring of the wetwell floor by enabling the creation of a hydraulic jump across the bottom of the trench. The hydraulic jump is created periodically with the use of an isolation gate located at the inlet of the wetwell. It is anticipated that this cleaning practice will be carried out on a weekly basis.

The width of the trench should limit maximum approach velocities to less than one fps. The spacing of the pumps will be determined by the manufacturer's recommendations. The pumps will be guide-rail mounted to allow their removal/replacement without entering the wetwell. Aluminum hatch covers with safety grates will provide access to the pumps. An appropriate spray-on, polyurea or epoxy coating system will be used to protect the wetwell from corrosion.

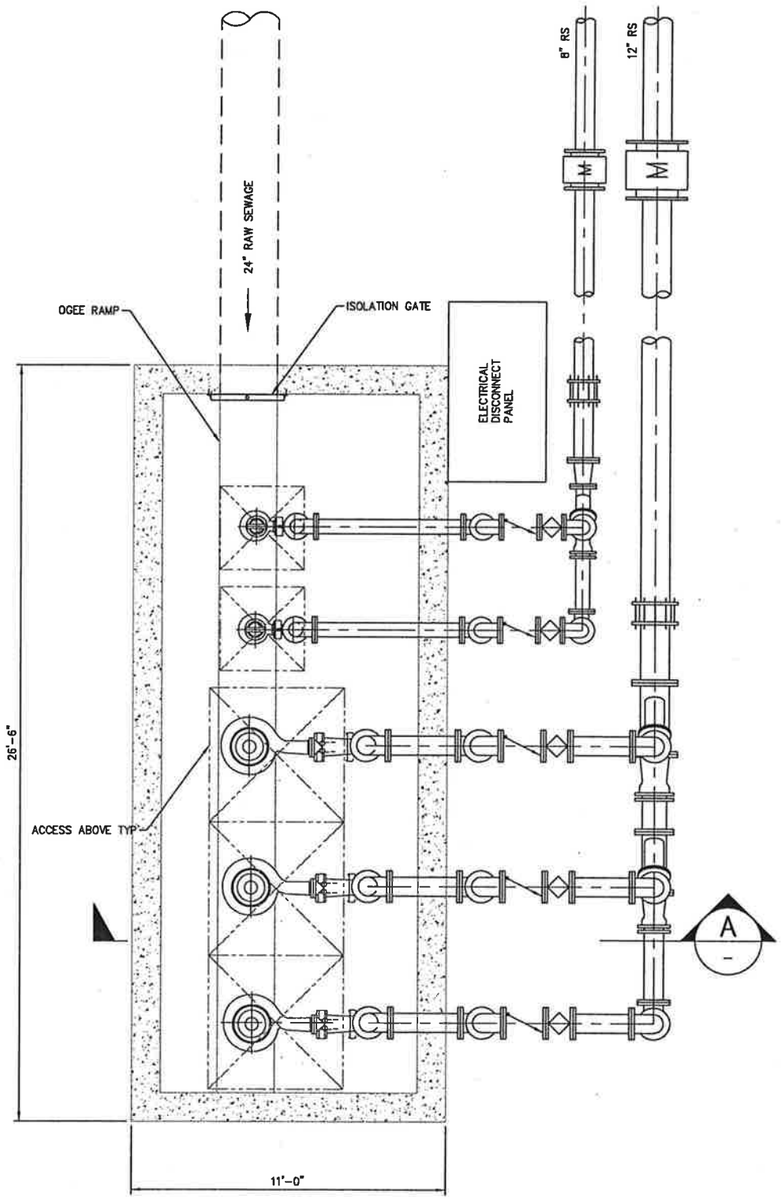
Each pump will have its own discharge pipe, a check valve, and an isolation valve. The valves will be located above ground for easy access and cost savings.

It is anticipated that dual forcemains will be required to develop adequate scouring velocity under low flow conditions and to optimize the head/flow characteristics of available pumps. It is anticipated that five pumps will be required consisting of two jockey pumps and three primary pumps. The pumps will be sized so that both "jockey" pumps and two primary pumps all pumping simultaneously will meet the peak instantaneous flow condition. The third primary pump is considered a "stand-by" pump to meet reliability requirements.

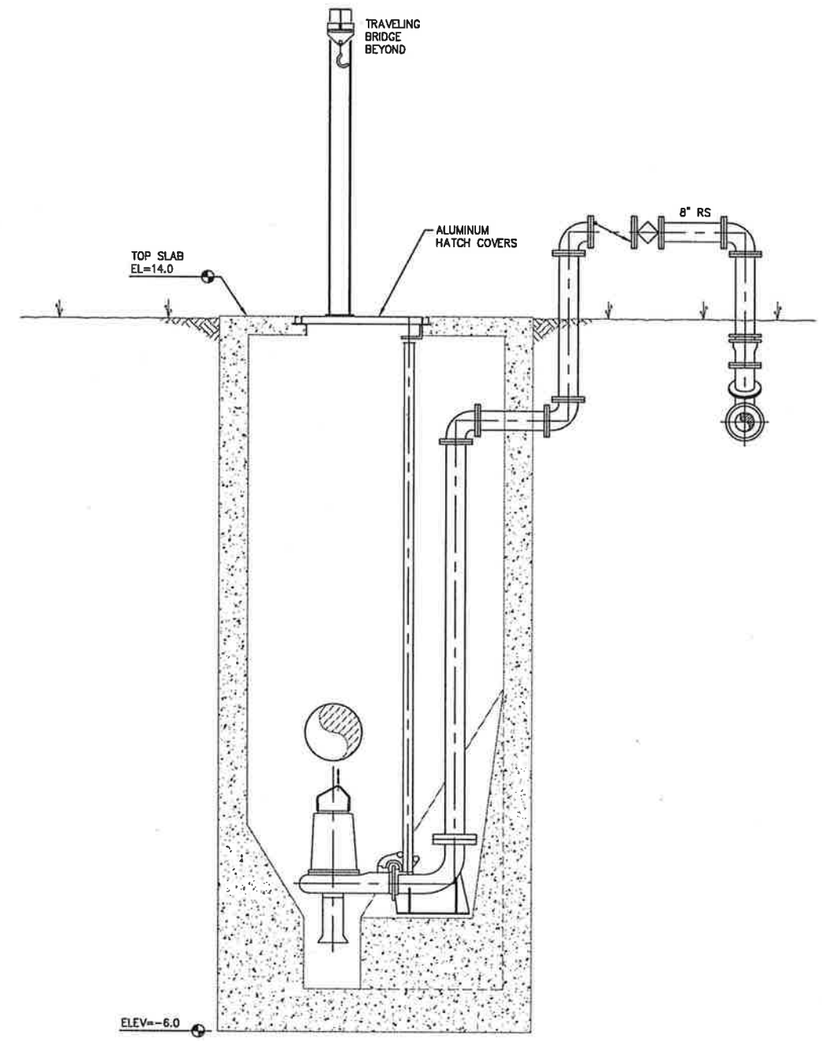
Discharge pipes will be manifolded together outside of the valve vault to reduce the size of the valve vault. The proposed layout for the station is shown on Figure 2.1.

Pump Extraction

The pump station will be equipped with an overhead bridge crane suitable for extracting and installing the pumps. The bridge crane will be configured to enable loading the pumps onto a pickup or flatbed truck for offsite maintenance procedures.



PLAN VIEW
SCALE: 3/16"=1'-0"



SECTION A
SCALE: 3/16"=1'-0"

Kennedy/Jenks Consultants

City of Tillamook

INFLUENT PUMP STATION
PLAN AND SECTION

0491010.20

FIGURE 2.1

Pump Controls

The pumps will be fitted with adjustable frequency drives to allow for the maximum coverage over the pumping range for each pump. Sensors for motor over-heating, seal leakage, and excessive amperage draw will be included. Each pump shall be equipped with a run time meter. A pressure transducer will be used to provide wetwell depth information to the controller. A conventional float system (or equivalent system) will be used as a backup to the pressure transducer, to detect emergency high and low water levels in the wetwell.

A PLC controller will be used to evaluate the inflow conditions and to call the appropriate pump(s) to run/stop. The PLC will also provide the required alarm functions. Other control panel components will be specified per standard Kennedy/Jenks Division 16 (Electrical) and Division 17 (Instrumentation) specifications. The location of the control panel is to be determined.

A disconnect panel, located close to the wetwell, will be provided so that pump power cords can be easily disconnected for pump removal. An air gap is required between this panel and the wetwell (i.e., no continuous conduit connection between the disconnect panel and the wetwell) to meet NFPA 820 requirements.

Additional Considerations

It is anticipated that Flygt ITT will be the basis of design. Kennedy Jenks' standard specification 11303 will be used to specify the pumps and appurtenances. The pump motors shall be explosion proof. All switches, junctions, and other electrical components located inside the wetwell must comply with NFPA 820 for potentially explosive environments. An annular ring style pressure gauge will be fitted to the discharge side of each pump.

Technical Memorandum No. 3

Subject: Headworks Improvements
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Harry Ritter, P.E.

Introduction

The existing headworks channel is integrated with the primary clarifier. This above-grade structure was constructed in 1998 and will be incorporated into the upcoming WWTP expansion. The headworks channel includes a Parshall flume with an ultrasonic level sensor, a Pista Grit™ grit removal system in a 360-degree configuration, a communitor, and influent sampling equipment. Raw sewage enters the upstream end of the channel through a 20-inch pipe. Process recycle flow (i.e., digester supernatant and secondary waste solids) enter the downstream end of the channel through a 4-inch pressure main. The headworks channel is covered with aluminum grating serving as a walkway surface. This walkway is approximately four feet above the clarifier deck. Access to the headworks from the clarifier deck is via two ship ladders. A plan view of the existing primary clarifier and integrated headworks is shown on Figure 3.1.

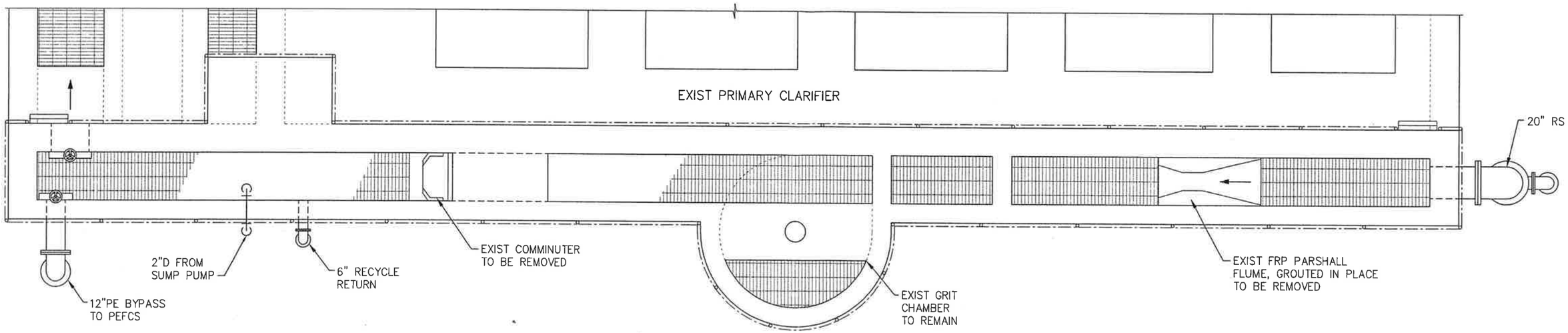
The invert of the headworks channel is roughly 12-feet above grade in the upstream reach of the channel. The channel drops approximately 18-inches as it passes through the grit chamber. Downstream of the grit chamber, the channel bottom reverts to the upstream elevation. At the downstream end of the channel, the invert drops 30-inches abruptly as the headworks transitions to the distribution channel of the primary clarifier. There is only one channel (no bypass channel).

The existing headworks channel is 3'-10" deep and 3'-0" wide for most of its length. The channel is deeper following the 30-inch invert drop noted above. The invert of the Parshall flume, installed in the upper reach of the channel, is approximately 19 inches above the channel floor, reducing the channel depth in that section to 2'-1". As part of the Corrective Action Project undertaken in 2003, the 9-foot channel section upstream of the Parshall flume was filled with grout to raise the bottom of the channel level with the floor of the flume. This was done to eliminate an ongoing maintenance problem caused by solids accumulation upstream of the flume. As a result of the elevated placement of the flume, the capacity of the channel is reduced to approximately 5.5 MGD. This estimate is based on actual observation of the water depth in the channel at this flowrate.

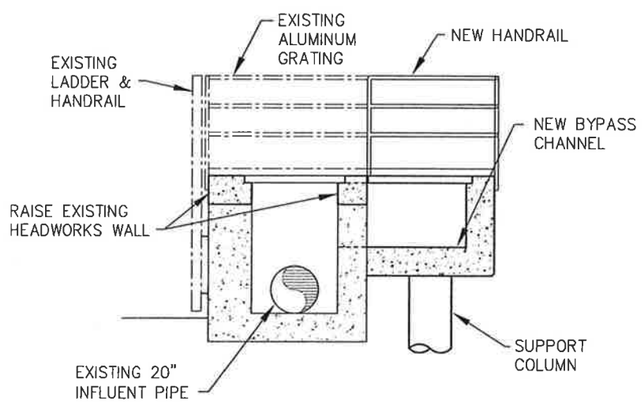
Planned Improvements

Currently, the headworks channel has inadequate hydraulic capacity for the future design flows, has no means to remove plastic and fibrous material from the influent stream, and has no bypass channel which is required to meet reliability requirements. To address these deficiencies, the following improvements are planned:

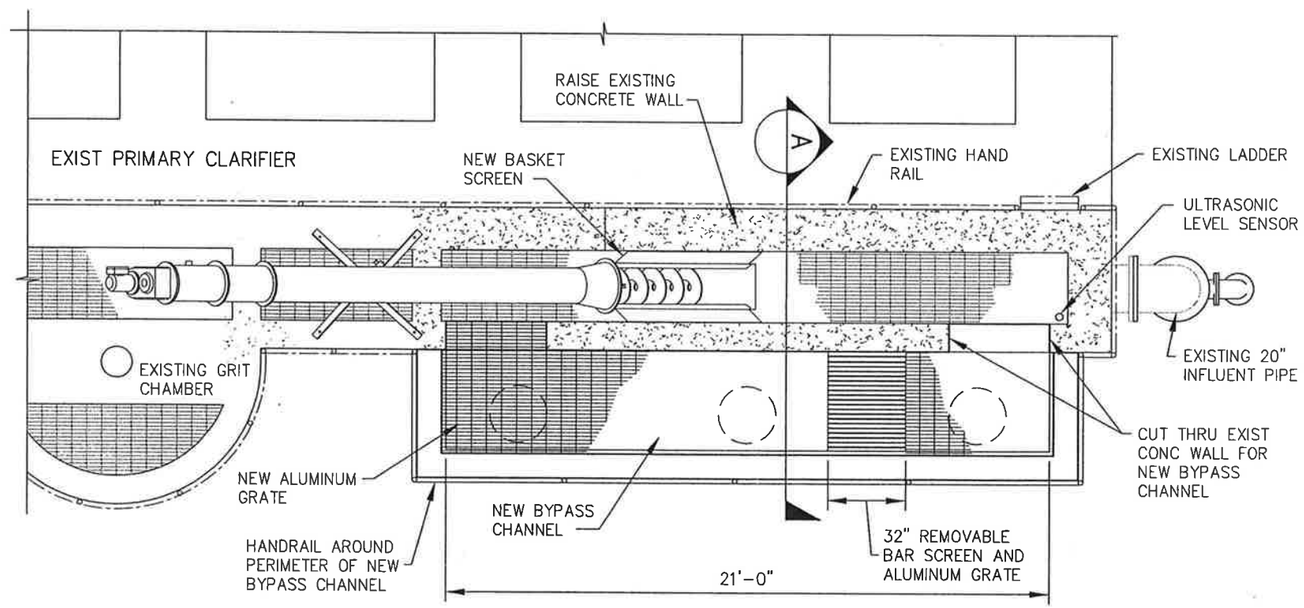
- removal of the existing Parshall flume



PLAN
EXISTING HEADWORKS
SCALE: 3/16"=1'-0"



SECTION A
SCALE: 3/16"=1'-0"



PARTIAL PLAN
MODIFIED HEADWORKS
SCALE: 3/16"=1'-0"

Kennedy/Jenks Consultants

City of Tillamook

EXISTING HEADWORKS AND PARTIAL PLAN OF PRIMARY CLARIFIER

0491010.20

FIGURE 3.1

- removal of the existing grout in the upper reach of the channel
- removal of the existing communitor and controls
- installation of a self-cleaning fine screen with dewatering auger and controls
- installation of a small jib crane to hoist the fine screen out of the channel
- modifications to the existing hand railing to allow the fine screen to be articulated from side to side on its pedestal mount
- construction of a bypass/overflow channel with a manual bar screen to divert flow around the primary channel
- extending the walls of the upstream reach of the headworks channel to increase the peak flow capacity.
- installation of new grating and hand railing to provide a walkway/maintenance surface on the new bypass channel
- replacement of one of the ship ladders providing access to the headworks walkway with an aluminum staircase for operator safety (this request was made by the DEQ).

Figure 3.1 illustrates many of these planned improvements. Based on the lower capital, installation, and O&M costs, the Facility Plan recommended a perforated basket type fine screen with an integrated screenings washing system and screw auger/compactor. Many manufacturers supply this type of headworks screen and they are very popular for small plants. These types of screening systems are generally pedestal mounted and can be lifted out of the channel with a small hoist for maintenance. The screened material is spray-washed as it moves up the auger to remove putrescible material. The screenings are compacted at the head of the auger before being discharged to a bag attachment used to collect the screenings.

Design Criteria

The following criteria will serve as the basis of design for the screening system and bypass channel design:

- The bypass channel should have sufficient capacity to meet the peak hour flowrate requirement of 10 MGD.
- The fine screen should be sized to meet the peak day flowrate of 7.2 MGD (this may not be possible due to physical constraints of the channel depth and equipment flow characteristics). The bypass channel will be designed to convey a portion of peak

instantaneous flowrate, or the total flow under conditions when the fine screen blinds and cannot be cleared due to equipment failure.

Design Considerations

- The floor of the bypass channel will be elevated above the floor of the primary channel so that under most conditions, the flow stream is not diverted through this channel (assuming the fine screen is clean). This will alleviate the need for flow isolation gates and will keep the channel dry when not in use.
- The overflow channel will include a manually cleaned bar screen.
- Since the primary channel can be temporarily “isolated” by shutting down the influent pumps and the fine screen can readily be lifted out of the channel for maintenance, flow isolation gates will not be required.
- Walkway grating and handrails on the new bypass channel will match existing materials.
- Provisions shall be provided to transfer compacted screenings from the elevated headworks channel to the ground surface.
- A dumpster should be provided for screenings.
- Freeze protection should be provided for the water supply to the screening washer.

The bypass channel will be constructed on the south side of the headworks channel and be supported by concrete columns. The width of the bypass channel should be set such that 100% of the peak hour flow can be conveyed through the channel. The channel should be slightly sloped so that it will drain once the water level in the main channel is below the invert of the bypass channel.

The screening system shall use an ultrasonic level sensor to monitor upstream water level and to signal the conveyor system to move material up the compaction zone. An auxiliary float will be installed as a backup to provide a HWL alarm to the SCADA system.

Technical Memorandum No. 4

Subject: Liquid Flow Metering
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Harry Ritter, P.E. and Tim Caire, P.E.

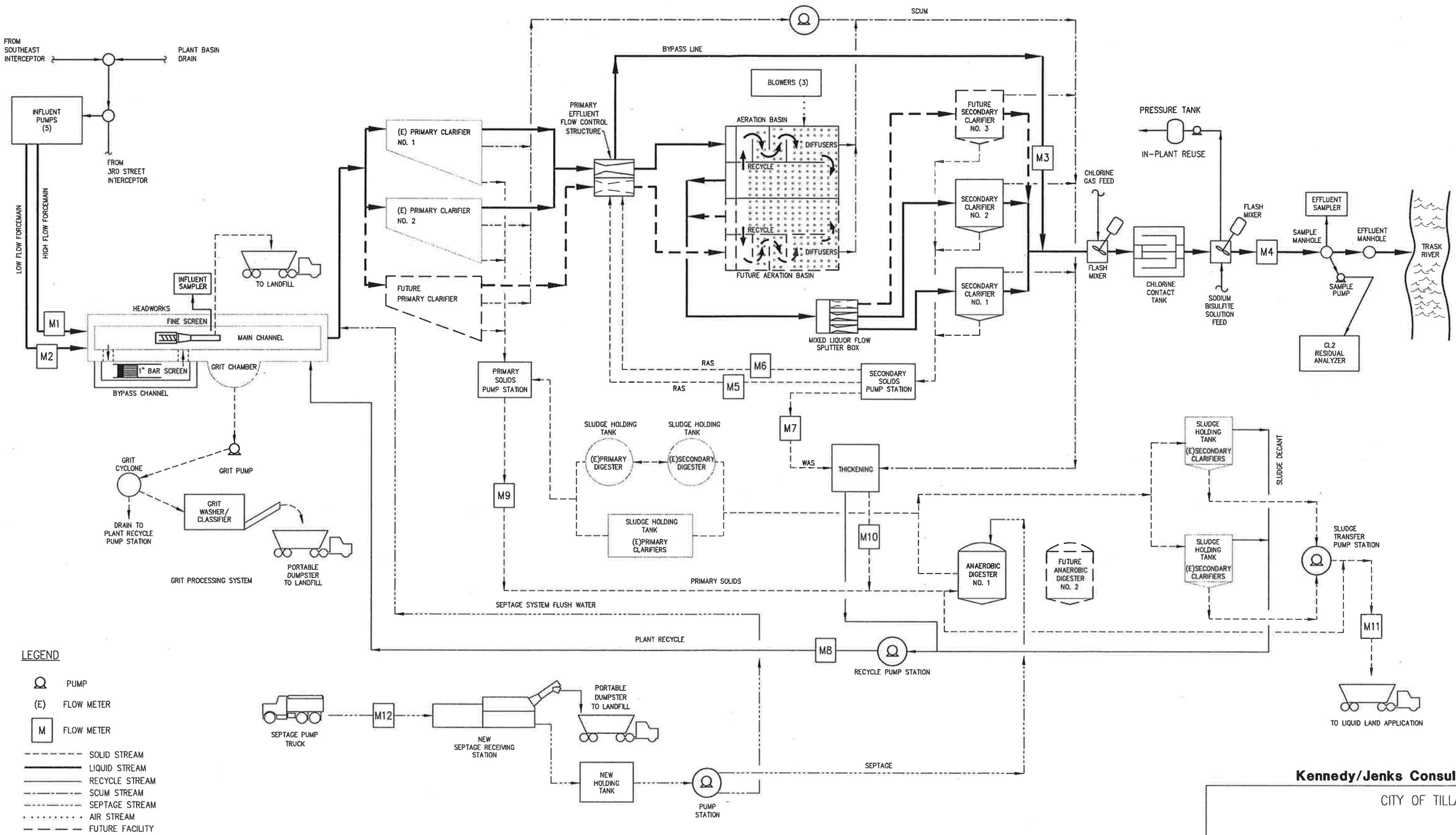
Introduction

The following liquid streams will need to be metered at the new WRF:

- Influent (meters M1 and M2)
- Peak Flow Bypass (meter M3)
- Effluent (existing meter M4)
- Return Activated Sludge (meters M5 and M6)
- Waste Activated Sludge (meter M7)
- Plant Recycle (meter M8)
- Primary Sludge (existing meter M9)
- Thickened Sludge Meter (meter M10)
- Sludge Transfer Pump Station (meter 11)
- Septage Receiving Station (meter 12)

With the exception of the peak flow bypass meter (M3) and the existing effluent meter (M9), all of the liquid flow metering proposed for this project will be electromagnetic type meters commonly referred to as “mag” meters. Mag meters are often the most cost-effective meter for the types of liquids commonly encountered in treatment plants. Mag meters are typically used on pressure lines but can also be used on gravity lines if the line is flowing full at all times. Normally, the electromagnetic sensors will be integrated into a short pipe barrel with flange connections on both ends. The barrel diameter is usually smaller than the carrier pipe to increase the velocity of the flow as it passes through the meter, as a mag meter must operate within a specified range of flow velocities (typically 1 fps to 30 fps) for repeatable accuracy.

Flow through the meter should be non-turbulent. For this reason, mag meters normally require at least ten straight and unobstructed pipe diameter equivalents upstream of the meter and at least five straight and unobstructed pipe diameter equivalents downstream of the meter to reduce turbulence. Mag meters normally are designed to transmit an analog signal proportional to flow. Materials of construction used for the sensor barrel, sensors, and electronics enclosure should be compatible with the liquid being metered and the environmental conditions. The meter will normally have a local display panel and be connected to the plant SCADA for control, monitoring, and reporting purposes. When appropriate, meters should be equipped with a reset



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PROCESS FLOW SCHEMATIC

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FIGURE 4.1

button to facilitate convenient reset of the meter. The locations of the meters are shown on the WWTP Flow Schematic, Figure 4.1.

Proposed Meters

Influent Flow Meter

The existing WWTP uses a Parshall Flume located in the headworks channel for influent flow metering. The flume reduces the cross sectional area of the channel and its peak flow capacity to approximately 5.0 MGD. Since the existing headworks will be incorporated into the expanded WWTP and the new influent pump station will be designed to pump up to 10 MGD, the Parshall flume must be removed. It is proposed that it be replaced by two in-line meters, M1 and M2, located on each of the discharge forcemains from the new influent pump station. Locating the meters on these discharge lines will satisfy the requirement to meter the influent flow in a location upstream of any plant recycle flows. The meters will be located adjacent to the existing Primary Clarifiers, above-grade and in a vertical orientation.

Peak Flow Bypass Flow Meter

Primary effluent bypass flow is defined as the flow that bypasses the secondary treatment system during peak flow events. Under peak flow conditions, the Primary Effluent Flow Control Structure (PEFCS) will bypass a portion of the primary effluent directly to the chlorine contact tank. Meter M3, consisting of an ultrasonic level indicator and sharp-crested weir will be used to meter this flow. Since all flow meters will be incorporated into the SCADA system, indirect calculation of flow to the secondary process can be achieved by subtracting the flow through the primary effluent bypass line (M3) from the influent flow (sum of M1 and M2).

Effluent Flow

Effluent from the existing WWTP is metered with an electromagnetic velocity/level sensor located in the discharge pipe just upstream of the effluent sample point. This meter (M4) was installed as part of the 2003 CAP and is planned to be reused. The meter uses an electromagnetic sensor attached to a stainless steel band that is sized specifically for the existing pipe diameter. Since this pipe will be replaced with a larger pipe to provide greater peak flow capacity, the existing sensor will need to either be modified or replaced.

Return Activated Sludge

Due to the large range of flows that the RAS pumping system must accommodate, it is anticipated that dual forcemains will be required. The forcemains will terminate at the Primary Effluent Flow Control Structure. Meters M5 and M6 will be used to meter the two force mains. Consideration should be given to locating the meters in a vertical orientation at the flow splitter structure to eliminate the need for a vault.

Waste Activated Sludge

WAS will be metered through meter (M7) fitted to the common discharge line from the WAS pumps located at each clarifier. It may be possible to install the meter in the Solids Processing Building. If not, a vault will be installed to house the meter. This meter will be used to control the thickening process.

Plant Recycle

The plant recycle meter (M8) will be fitted to the discharge line of the Recycle Pump Station. This meter will record digester supernatant and WAS thickener filtrate flows.

Primary Sludge

Primary Sludge will continue to be metered using the existing mag meter (M9) located in the pump room of the primary clarifier. This meter was installed as a part of the 2003 CAP.

Thickened Sludge Meter

Thickened sludge will be metered using meter M10 fitted to the discharge line of the thickened sludge pump. This meter will be located in the Solids Processing Building.

Sludge Loading Dock Meter (M11)

This meter will record sludge pumped from all sources to the sludge loading dock.

Septage Receiving Station (M12)

This meter will record the volume of septage delivered to the septage receiving station.

Technical Memorandum No. 5

Subject: Flexible Aeration Basin, Blowers, and Air Piping
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Tim Caire, P.E. and Harry Ritter, P.E.

Introduction

The Phase One Flexible Aeration Basin (FAB), which will consist of two adjacent tanks, will provide secondary treatment. During design, consideration will be given and provisions will be included, where applicable, for adding two additional tanks in the future. The FAB will be preceded by primary clarification and will be followed by two conventional secondary clarifiers. A hydraulic control structure will be incorporated in the yard piping to enable partial bypass of the FAB under peak flow conditions, as described in Technical Memo No. 9. The FAB will be designed to remove BOD and to partially nitrify the ammonia load.

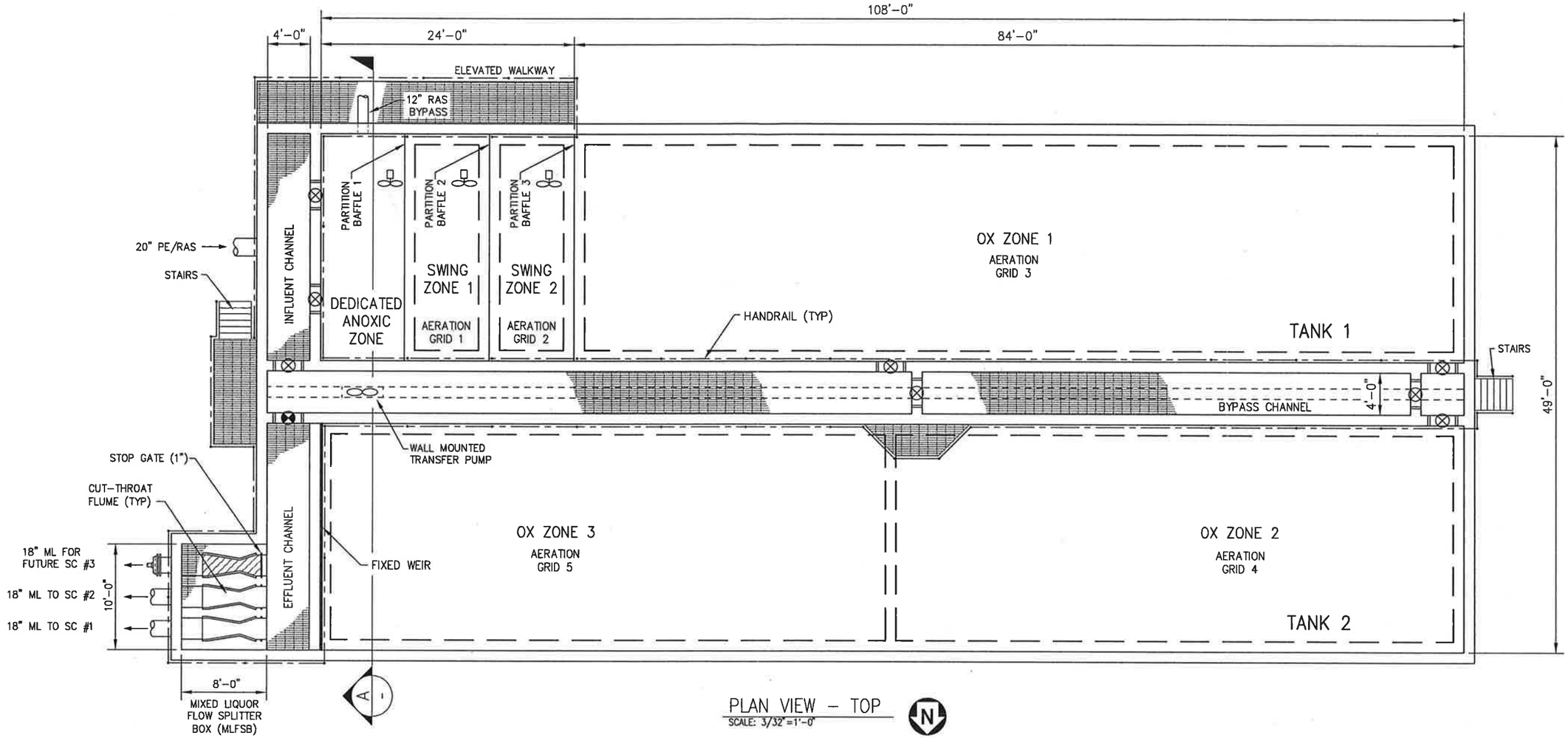
Three selector zones will be incorporated to discourage the growth of filamentous bacteria. The selector zones will occupy approximately 15% of the total basin volume. The FAB will be based on the Modified Ludzack-Ettinger (MLE) activated sludge process, described below. In addition to selecting for settleable bacteria, the MLE process can provide partial recovery of oxygen and alkalinity which are consumed by the nitrification process.

The volumes of the aerobic and anoxic compartments, the blower selection, the aeration diffuser density, and hydraulic capacity of the flow control structures will all be verified during final design. A computer model will be used to simulate the process and confirm preliminary design estimates. Hydraulic capacity and elevation requirements of the flow control structures will be finalized also.

This Technical Memo describes the FAB, as well as associated blowers and air piping. Table 5.1 lists basic FAB Design Criteria.

Table 5.1 – FAB Design Criteria

Item	Design Value	Unit
Number of Tanks	2	
Basin Length	108	ft
Tank Width (each)	24	ft
Maximum Side Water Depth	20	ft
Minimum Freeboard	2	ft
Volume per Channel	0.389	MG
Total Volume	0.778	MG
Oxic Zone Volume	0.661	MG
Total Selector Zone Volume (3 zones)	0.117	MG
MCRT [a]	10	days
Organic Loading Rate	16 – 33	lb BOD/1,000 cf-day
MLSS Range	1,500 to 3,500	mg/l
Average Month Influent BOD_5	2,100 (1,470) [b]	ppd



PLAN VIEW - TOP
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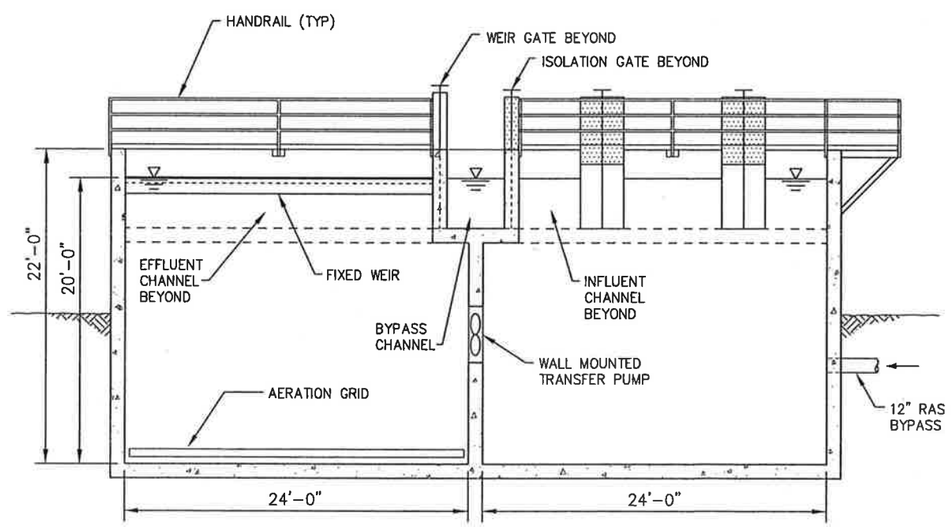

SECTION A
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LEGEND

- ISOLATION GATE (PLAN VIEW)
- ISOLATION GATE (SECTION VIEW)
- GRATE COVER
- WEIR GATE (PLAN VIEW)
- WEIR GATE (SECTION VIEW)
- SUBMERSIBLE MIXER
- HAND RAIL

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Aeration Basin (AB) and Mixed Liquor Flow Splitter Box (MLFSB) Top Plan and Section

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Figure 5.1

P:\cad\04\0491010.20_WWTP_DESIGN\Final Predesign Fig 2006\Fig 5-1 Aeration Basin.dwg Feb 01, 2006 - 2:40pm

Item	Design Value	Unit
Peak Month Influent BOD_5	2,475 (1, 733) [b]	ppd
Peak Week Influent BOD_5	4,200 (2,940)[b]	ppd
Effluent BOD_5 Limit	20	mg/l
Average Month Influent Ammonia	225	ppd
Peak Week Influent Ammonia	450	ppd
Effluent Ammonia Limit	10	mg/l
ADWF	0.9	MGD
AWWF	2.5	MGD
MMWWF	4.0	MGD
PDF	5.0 [d]	MGD

a. MCRT for Peak Month BOD_5 Loading and MLSS of 3,500 mg/l. Assumes overall yield of 0.7 lb BOD/lb MLSS.
b. Value shown is for year 2025. Value in parenthesis is the startup value. Following 30% BOD removal in Primary Clarifier.
c. Value shown is for year 2025. Value in parenthesis is the startup value.
d. Calculated PDF = 7.2 MGD. Flows in excess of 5.0 MGD will be diverted around the FAB.

Basin Layout

A plan view of the FAB is shown on the Aeration Basin Plan, Figure 5.1. The two tanks will share a common wall and normally will be operated in series. A center channel, mounted atop the common wall, will enable either tank to be independently taken off-line for maintenance. Influent and effluent will enter and leave at the same end of the basin.

Primary effluent and RAS will enter Tank 1 through the influent channel upstream of the first anoxic zone. Normally, flow will pass from the influent channel to the dedicated anoxic zone and then through two downstream zones termed "swing zones". The two "swing zones" will contain independent aeration grids and be capable of operating as anoxic selectors or as aeration zones. The swing zones, along with the center channel and flow isolation gates, will provide a high degree of flexibility in flow path and treatment process.

At the downstream end of Tank 1, mixed liquor will flow into Tank 2 through gates located in the center channel. A wall pump, located in the downstream end of Tank 2, will return nitrified mixed liquor into the first anoxic zone of Tank 1 where it will combine with the primary effluent and RAS. The wall pump will be variable speed and should provide for a recirculation rate of 150% of the influent flow rate. Effluent will leave the FAB through the effluent channel and will be split by flumes, as discussed in Technical Memo No. 9. Provisions will be incorporated for future basin expansion.

Activated Sludge Process

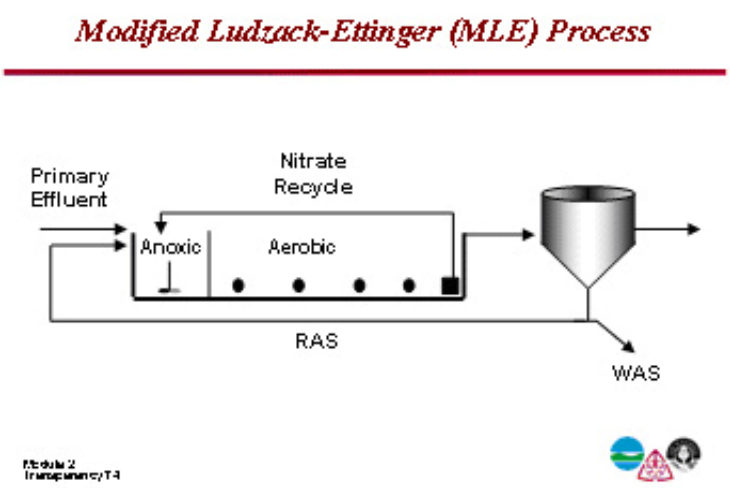
The MLE activated sludge process will be utilized, as noted above. A schematic of this process is provided in Figure 5.2. This process is capable of both nitrification and denitrification. As

discussed above, this process was selected for the expanded Tillamook WWTP because it can provide nitrification of ammonia, and because it selects and favors growth of non-filamentous bacteria with favorable settling characteristics. With adequate anoxic volume, the MLE process can also provide partial or full denitrification. For the Phase One FAB, the anoxic zones (i.e., selectors) are only intended to enhance selection of non-filamentous bacteria. If future permit limits require enhanced denitrification, it could be incorporated into the FAB by increasing the volume of the anoxic zones under the Phase Two expansion.

Mixed liquor flows from the anoxic selector zone(s) to the aerobic zones of Tank 1 and Tank 2 where nitrification occurs. A fraction of the mixed liquor (up to 150% of the influent flow rate) from Tank 2 is recycled to the selector at the upstream end of Tank 1. RAS and Primary Effluent are mixed with the nitrified mixed liquor in the first selector zone. As a result, “selection” of non-filamentous microbes is enhanced due to both the anoxic environment and the high ratio of food to mass (F/M) in the selector zone. Both of these conditions favor the growth of floc-forming bacteria over filamentous bacteria. Additionally, the upstream location of the anoxic zone promotes denitrification because it ensures that the anoxic zone will receive a source of readily-biodegradable substrate; this precludes the need for the addition of an external carbon source such as methanol.

In addition to the MLE process, the basins will be designed to allow for operation in a “step-feed” or “sludge re-aeration” mode. The sludge re-aeration mode can be utilized during periods of peak inflow to conserve solids. While in this mode Tank 1 receives only RAS, and PE is introduced at the midpoint or downstream end of Tank 1. Washout of solids from the tanks is minimized.

Figure 5.2 – Modified Ludzack-Ettinger (MLE) Process



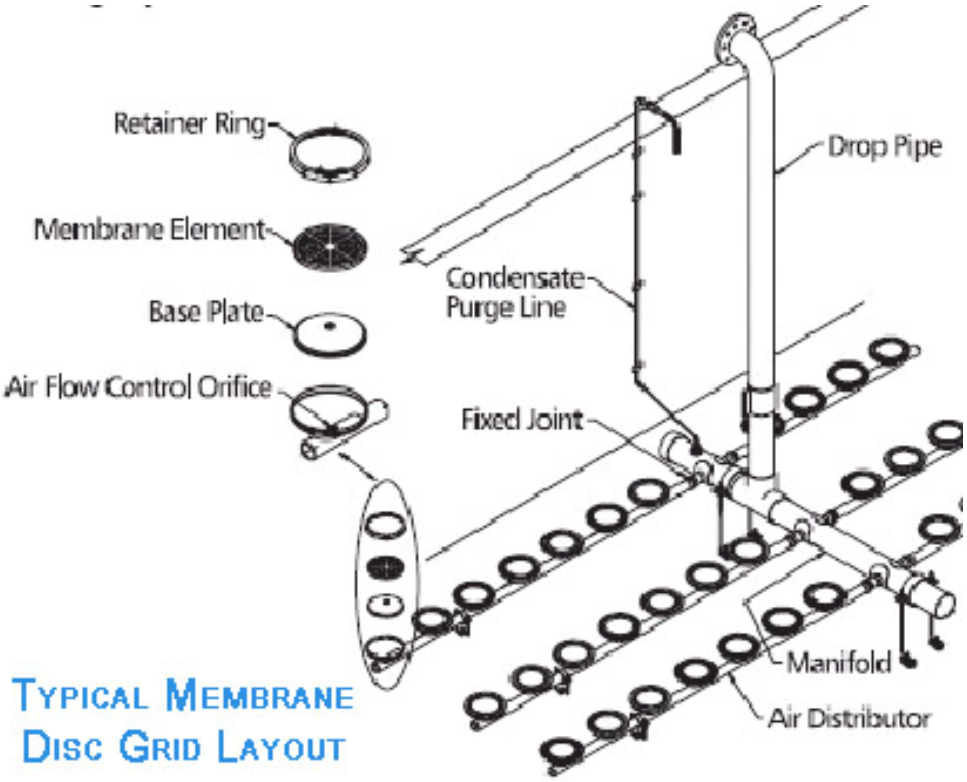
Air Piping and Diffusers

Air will be routed from the new Blower Building to the FAB through welded stainless steel piping. An alloy of Type 304 and a class of Schedule 10 will be used. Pipe diameters will likely range from 4-inches to 10-inches. A detailed air piping analysis will be conducted during final design.

Air will exit the delivery system and enter the mixed liquor via membrane-disc fine-bubble diffusers. These diffusers provide high oxygen transfer efficiency and induce a relatively low drop in system air pressure. The grid system which supports the diffusers will be constructed of PVC. A typical diffuser system (by Sanitaire) is illustrated in Figure 5.3.

Input regarding optimal diffuser floor coverage and number of diffuser zones shall be based on the result of the process modeling and per recommendations of the diffuser manufacturer. Dissolved oxygen monitoring in conjunction with variable speed drives on the blower motors will be used to control the air supply. Typically, a minimum dissolved oxygen concentration of 2.0 mg/l will be maintained in first stage of the aeration basin. The diffuser grid coverage is normally reduced at the downstream end of the process.

Figure 5.3 – Typical Membrane Disc Grid Layout



Blowers

Preliminary calculations indicate that three identical 60-Hp blowers will be provided to serve the expanded Tillamook WWTP. One of the units will normally serve as a standby unit. Two blowers operating simultaneously should have the capacity to satisfy the peak week average demand. Under unusual conditions, all three blowers should have the flexibility of being placed into service simultaneously. Blower capacity will be based upon design BOD_5 and ammonia loadings shown in Table 5.1.

Centrifugal blowers would provide required air and pressure delivery, turndown capability, and low noise emission. An alternative to centrifugal blowers is rotary lobe blower. Newer, sound-attenuated versions of the rotary lobe blower can offer several significant advantages to centrifugal units, including:

- lower capital cost
- low noise emissions
- high efficiencies
- wide range of turndown capability

Turndown capability will be provided through the use of variable frequency drives or, if centrifugal units are used, a throttled butterfly valve on the suction side of the blower.

Technical Memorandum No. 6

Subject: Secondary Clarification
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Harry Ritter, P.E., and Tim Caire, P.E.

Introduction

Secondary clarification removes the biomass associated with secondary treatment producing a clarified effluent prior to disinfection and final discharge. This unit process also serves to concentrate biomass in the mixed liquor so that it can be recirculated to the biological treatment process or wasted to the digester for subsequent processing. The recirculated stream is referred to as return activated sludge (RAS) and the wasted portion is referred to as waste activated sludge (WAS).

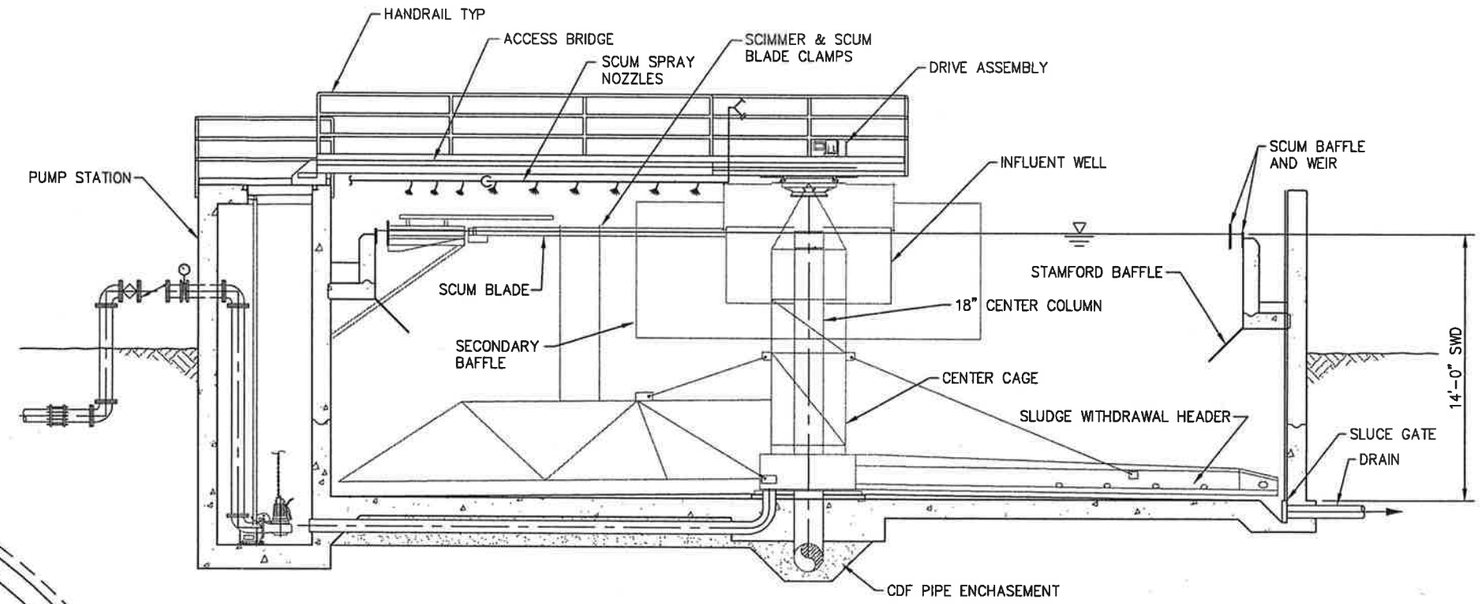
Secondary clarifiers must be large enough to provide sufficient residence time for the solids to settle and not be swept upward and over the weir. The residence time is a function of the "overflow rate" which is an expression of hydraulic loading. Overflow rate is defined as the daily flow in gallons per day divided by the surface area in square feet (gpd/sf). The overflow rate can also be expressed as the average upward velocity of the liquid in the tank (ft/day). For sedimentation to occur, the settling velocity of solids (particles) must be greater than the overflow rate.

A rapid-return clarifier is proposed for this project based on the efficiency, flexibility and proven success of this design. This type of clarifier draws solids from the bottom of the clarifier into the rotating collection header. The collection header can be directly piped to the RAS/WAS pumps or the header can be hydraulically connected to a wetwell being served by submersible RAS/WAS pumps. A wet well with submersible pumps will be used for this project, as described in Technical Memo No. 7.

A typical rapid-return clarifier is shown on Figure 6.1. Mixed liquor from the aeration basin is introduced into the feedwell through inlet ports in the center column. From the feedwell, the mixed liquor travels downward, under the secondary baffle which serves to distribute the flow evenly and disperse any turbulence or eddy currents. As the liquid moves from the center column to the periphery of the clarifier, suspended solids settle to the clarifier floor. The clarified liquid flows over the effluent weir and into the launder. Accumulated solids are drawn into the sludge collection header which rotates slowly along the floor of the basin. The scum collection mechanism is connected to the rotating clarifier mechanism and deposits scum and other floating material into the scum box once per revolution.

Design Criteria

Parameters that must be defined or checked for the design of secondary clarifiers include the hydraulic overflow rate, the solids loading rate, the side water depth, and the weir loading rate. In addition, reliability criteria must be met which may impact the size and number of clarifiers.



SECTION A
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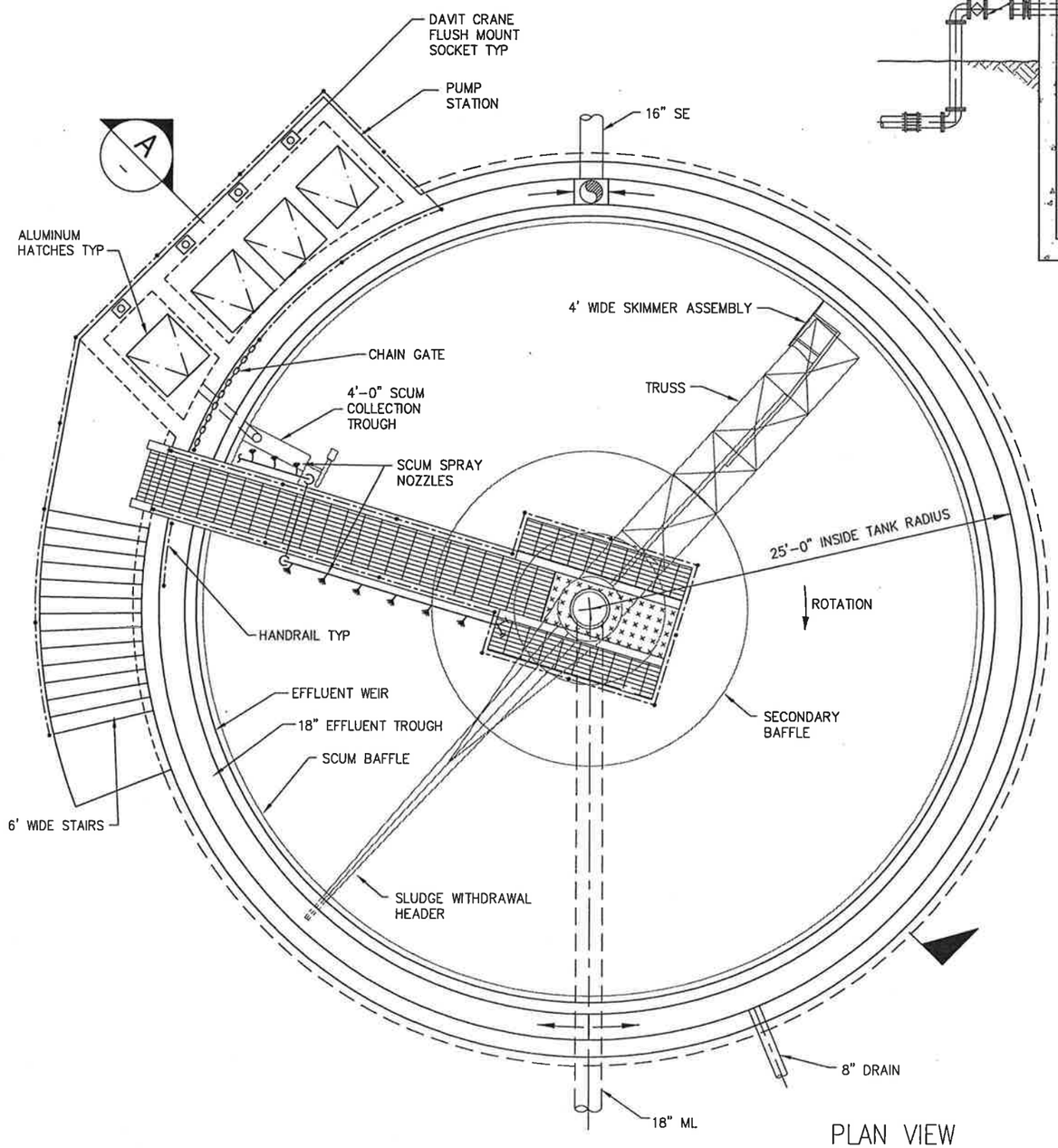

PLAN VIEW
SCALE: 1/8"=1'-0"

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City of Tillamook, Oregon

SECONDARY CLARIFIER PLAN AND SECTION

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Figure 6.1

- Properly designed and equipped secondary clarifiers may operate satisfactorily at a peak overflow rate of 2,000 gpd/sf or greater depending on the settling characteristics of the sludge, the depth of the clarifier, and the type of sludge collection mechanism. Overflow rates typically range from 500 to 800 gpd/sf during average plant flowrate and from 1,000 to 1,500 gpd/sf during peak flowrate. If information is not available on sludge settleability, a more conservative value of 1,250 gpd/sf is recommended as the peak hydraulic loading rate.

- Allowable solids loading rates may range between 0.8 to 1.2 ppd/sf for average loading conditions. A value of 2.0 ppd/sf is allowable under peak loading conditions.

- Recommended side water depth depends on the type of biological process preceding clarification. Depths for an activated sludge process typically range from 12 to 16 feet. In general, increased depth will allow for better clarification under peak flow conditions by keeping the top of the sludge blanket below the hydraulic influence of the effluent launders. According to equipment manufacturers, rapid-return clarifiers with “active” sludge collection function adequately with less depth than older, more conventional designs. A depth of 14 feet is proposed for the new clarifiers.

- Allowable weir loading rates range from 10,000 to 30,000 gpd/ft. For weirs mounted to the outside wall of a round clarifier, the weir loading rate will always be within this range, providing the overflow rate is within the acceptable range

Table 6.1 lists the hydraulic overflow and solids loading rate for the two 50-foot clarifiers proposed for this project. The design peak flow rate (not including RAS flow) is 5.0 MGD. Influent flowrates in excess of 5.0 MGD will be bypassed around the secondary treatment system.

Table 6.1 – Hydraulic Overflow and Solids Loading Rates

Flowrate (MGD)	Flow Condition	One Clarifier in Service [a]		Two Clarifiers in Service [a]	
		Overflow Rate (gpd/sf)	Solids Loading (ppd/sf)	Overflow Rate (gpd/sf)	Solids Loading (ppd/sf)
0.90	ADWF	460	0.62	NA	0.31
4.0	AWWF	2,040	0.62	1,020	0.31
5.0	PDF [b]	2,550	0.62	1,270	0.31

a. Based on 50-foot clarifier diameter. Solids Loading rate is based on Peak Month design loading. RAS recycle flows not included in values presented for overflow rate or solids loading.
b. Flows in excess of 5.0 MGD will bypass secondary treatment.

The 50-foot clarifiers proposed for this project are sized to accommodate the peak hydraulic loading. The solids loading rate is well below the allowable value for peak loading. The solids loading is independent of hydraulic loading due to the large degree of inflow and infiltration present in Tillamook's collection system.

Additional Design Considerations

Reliability Requirements

With regards to final sedimentation, EPA reliability requirements for Class I facilities state: "There shall be a sufficient number of units of a size, such that with the largest unit out of service, the remaining units shall have a design flow capacity of at least 75% of the total design flow to that unit operation". With one clarifier out of service, the remaining clarifier would have an overflow rate of 1,575 gpd/sf at 75% x PDF. This overflow rate is within the allowable range for peak flows. Therefore, the clarifiers sized for this project meet the EPA reliability requirement. Under normal conditions, it is assumed that a clarifier would only be taken off-line under low-flow conditions and that two clarifiers would normally be available under peak flow conditions. This is a reasonable assumption since the peak flow months are predictable at Tillamook.

Materials of Construction

The use of stainless steel for the clarifier mechanism and aluminum for the walkway and bridge will be evaluated during the design phase. These materials would offer the benefits of corrosion resistance and the elimination of coating maintenance over the lifetime of the facility. During final design, a cost evaluation should compare these non-corrosive materials versus traditional coated steel. A decision on which materials to specify will be made on the outcome of the comparison.

Miscellaneous Items

A spray water system will be attached to the underside of the access bridge to aid in the control of foam. The spray will guide foam to a scum hopper, which will be flushed periodically.

The effluent launder should be wide enough to allow maintenance personnel to walk within it during cleaning operations. The launder and the upper two feet of the waterline should receive an epoxy or ceramic coating to reduce the ability of algae to attach and to facilitate easier cleaning.

The walkway surface will be extruded aluminum or FRP grating to match the other walkway surfaces in the WWTP. Handrailing will be welded aluminum to match the other handrailing in the WWTP. Handrailing should incorporate a kick plate in accordance with code requirements.

Technical Memorandum No. 7

Subject: RAS/WAS/Scum Pumping
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Tim Caire, P.E., and Harry Ritter, P.E.

Introduction

Return of activated sludge, which is settled within the secondary clarifiers and pumped to the aeration basins, is an integral side stream within any activated sludge treatment scheme. Return activated sludge (RAS) provides multiple benefits including:

- Maintenance of desired range of concentration of biomass within aeration basins, despite large variations in plant influent flow rate,
- Maintenance of desired sludge layer in secondary clarifiers and thus control of solids carryover in clarifier effluent.

Activated sludge must also be removed from the main process flow in order to maintain the desired range of mean cell residence time (MCRT) within the aeration basins. The appropriate MCRT for the Modified Ludzack-Ettinger (MLE) process to be used at the expanded Tillamook WWTP is approximately 10 days.

This technical memorandum (TM) considers design elements associated with RAS and waste activated sludge (WAS) collection and delivery. The TM also discusses pumping of scum from the secondary clarifiers to the solids thickening equipment.

RAS Flow Rates

The flow rate of RAS will vary within a range which is defined by the desired recirculation ratio and the plant influent flow rate (the recirculation ratio is the ratio of RAS flow to plant inflow). For the Tillamook WWTP a recirculation ratio of 25% to 150% of plant inflow is recommended during dry weather conditions, and a recirculation ratio of 50% to 100% of plant inflow is recommended during wet weather conditions. The average dry weather flow rate is 0.9 MGD; therefore, the corresponding RAS flow rate will vary between 0.45 MGD and 1.35 MGD. For simplicity, these values are approximated as 150 gpm and 950 gpm. Because any flow rate greater than 5 MGD will bypass secondary treatment, the peak wet weather inflow and thus peak return flow rate is 5 MGD. Therefore, the flow range of the RAS system should be from 150 gpm to 3,500 gpm. Control of RAS flow is discussed below.

In order to achieve this entire range of return flow rates, a small and large pump will be installed. The RAS pumps will be coupled to variable frequency drives (VFDs), which will allow for speed reduction such that a turndown of at least 3:1 can be accomplished.

Pump Selection

Two smaller pumps (S1 and S2) and two larger pumps (L1 and L2) will be provided for the two proposed clarifiers. Assuming a turndown ability of 3:1 and with conservative estimates of additional head loss and corresponding flow reduction with two pumps in concurrent operation, the flowrates shown on Table 7.1 are estimated:

Table 7.1 - RAS Pumping Capacity

Pumps in Service	Delivery at Full Speed (gpm)	Delivery at 3:1 Turndown (gpm)
S1 or S2	450	150
S1 + S2	750	250
L1 or L2	1350	450
L1 + L2	2300	800
S + L [1]	1800	600
S1 + S2 + L1 + L2	3050	1050

[1] Independent RAS discharge piping will be provided for small pumps and large pumps (2 pipelines to the flow control structure).

Total dynamic head (TDH) requirements for the RAS pumps can be estimated by evaluating static lift and friction losses through RAS yard piping.

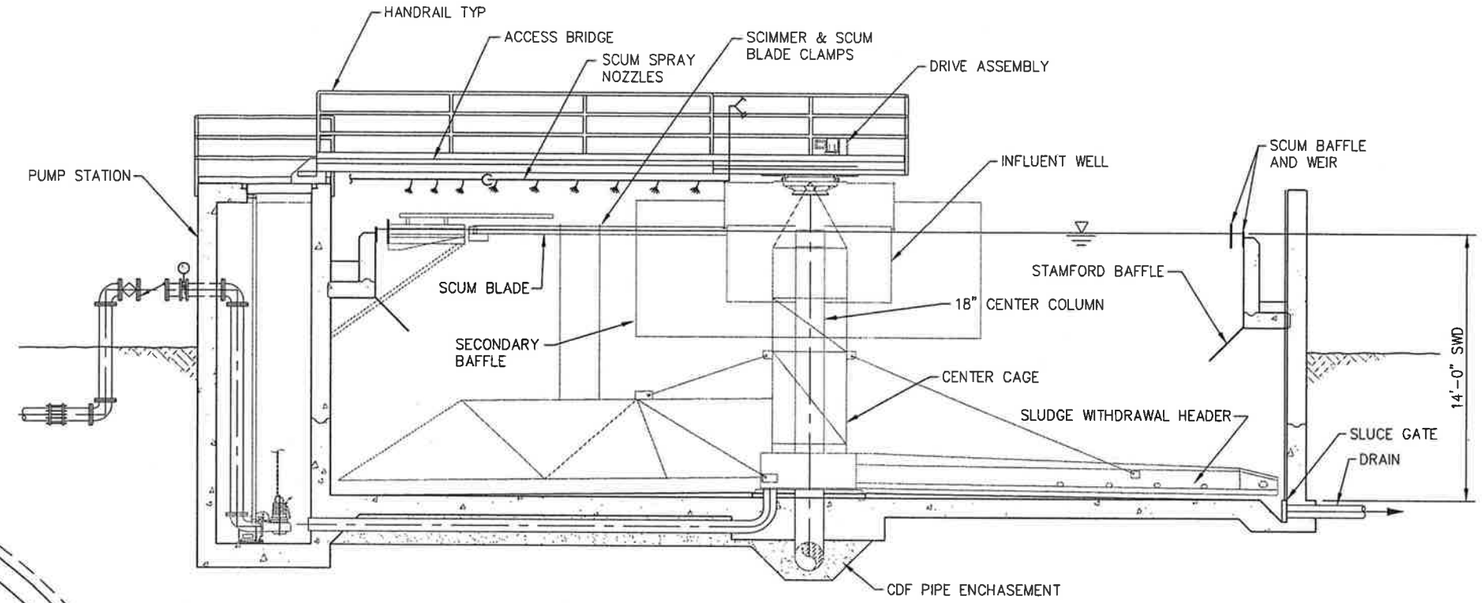
Static lift can be estimated by comparing the design water level in the flow control structure to the design water level in the RAS/WAS wet well. The former elevation is 24.5 and the latter elevation is 18.5, therefore static lift is estimated as six feet. Both RAS pipelines will be sized for a maximum velocity of seven feet/second and their anticipated lengths will not exceed 300 feet; the estimated maximum dynamic head loss is seven feet, including valve losses. Therefore, the estimated maximum TDH is 13 feet. These values will be confirmed during final design.

Pump Station Design

Design Requirements

Design requirements for the RAS/WAS pump station, in addition to delivery of the flow ranges stated above, consists of these criteria:

- collect and contain clarifier underflow
- meet layout, submergence, and cycle time requirements for RAS/WAS pumps
- accommodate scum pump(s)
- provide uniform RAS flow rate to aeration basins in order to minimize process impact
- minimize capital cost
- accommodate site layout constraints



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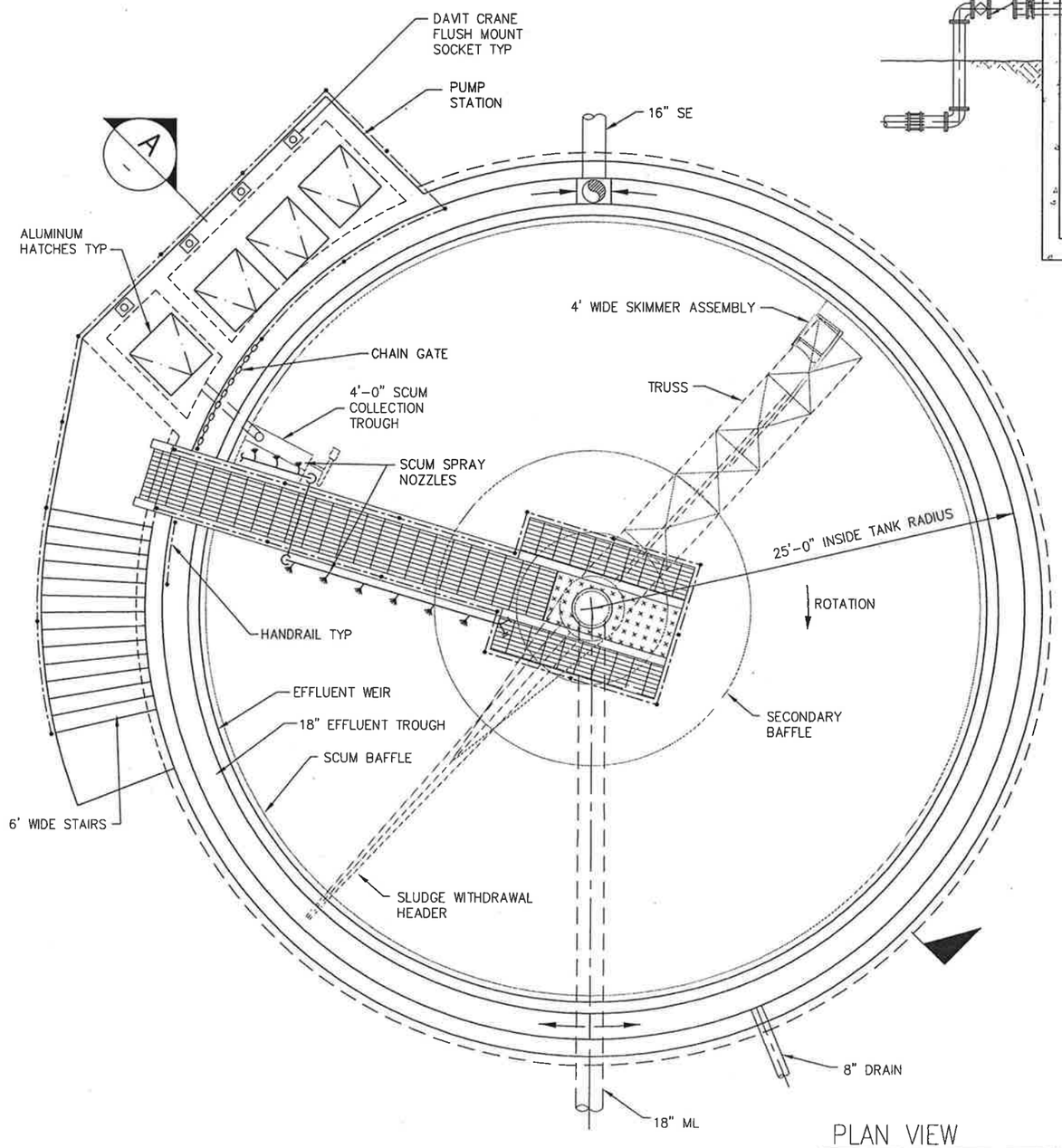

PLAN VIEW
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City of Tillamook, Oregon

SECONDARY CLARIFIER PLAN AND SECTION

0491010.20

Figure 7.1

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Pump Station Alternatives

The two most common arrangements for secondary sludge pumping use either vertical non-clog centrifugal pumps located within a specially designed pump room or submersible pumps located within a wet well that are directly connected to the clarifier underflow piping. While the vertical non-clog pumps are often favored from an operation and maintenance standpoint, the capital cost required to construct this type of pump facility far outweighs that of a submersible pump station. For this reason, this project will utilize a submersible pump station design.

A representation of the integral pump station design is provided in Figure 7.1.The station would feature one small pump RAS pump (S) and one large RAS pump (L) in a wet well constructed as part of the clarifier structure.

Pump Type and Preliminary Sizes

Submersible, nonclog, centrifugal pumps are commonly used for RAS and WAS pumping and are recommended for this design. A preliminary evaluation indicates that the smaller RAS pumps will require approximately 5-Hp motors and the larger RAS pumps will require approximately 15-Hp motors. WAS pumps will utilize 2-Hp motors or smaller (see WAS Design Considerations below).

Pump Control

The RAS pumps will operate continuously. By avoiding control via on/off setpoint water levels, RAS delivery will avoid intermittent operation. In addition, submergence and cycle time requirements are met more easily or are eliminated as considerations.

RAS pumps will be controlled primarily by flow pacing to plant influent flow rate. A secondary means of pump control will be linked to sludge blanket elevation in the clarifiers, with the objective of maintaining a constant blanket level.

WAS Design Considerations

WAS pumps will be placed adjacent to RAS pumps, within the same wetwell. WAS will be transferred directly to thickening equipment for processing; batch storage of WAS will not be provided in order to reduce capital costs.

This approach requires that the rate of WAS pumping match both the capacity of thickening equipment and also the intended operational practice for this equipment. Preliminary estimates of waste solids production and concentration indicate that, if thickening equipment will be operated for six hours per day at a frequency of five days per week, then the required WAS flow rate will be approximately 50 gpm. A reduced frequency of three days per week requires a WAS flow rate of approximately 80 gpm to 90 gpm. This represents the lowest recommended

frequency of WAS pumping, because if a reasonably-uniform MCRT is to be maintained, then time intervals between periods of solids removal cannot be excessive.

The smallest available capacity for gravity belt thickeners from leading manufacturers' ranges from approximately 50 gpm to 150 gpm; therefore, the intended maximum rate of WAS pumping of 90 gpm matches well with thickening equipment capacity and expected operational practice.

Scum Collection and Pumping

The ability to collect scum from the secondary clarifiers and the new aeration basin will be incorporated. When a filamentous bacteria problem arises in the aeration basin, it manifests as foam in the aeration basin. It can be beneficial to remove the scum from the aeration basin to the extent possible rather than transferring it into the clarifier. Scum from these three sources will flow by gravity to a scum collection wetwell integral to the RAS/WAS pump station wetwell. These gravity lines will be constructed with at least a one percent slope. They can be flushed using local hose bibs. Scum will be delivered to the WAS thickener.

One scum pump will be provided per wetwell. The scum pumps will be submersible and will provide a period of recirculation of wet well contents prior to delivery. They will also feature a grinder. A preliminary estimate of motor size is 5-Hp.

Yard Piping

The RAS system will contain two independent forcemains for delivery of RAS from clarifier wetwells to the flow control structure. A 6-inch diameter pipe will collect discharge from the smaller pumps, and a 12-inch pipe will serve the larger pumps. This approach ensures that an adequate scouring velocity will be maintained, and that the maximum velocity will not be excessive.

WAS and scum forcemain piping is sized at four inches in diameter. A larger diameter of six inches will be used for scum piping used for gravity flow.

Technical Memorandum No. 8

Subject: Chlorine Contact Tank and Disinfection Improvements
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Harry Ritter, P.E., Tim Caire, P.E.

Overview

This Technical Memorandum describes the new chlorine contact tank (CCT) and the proposed improvements to the existing chlorine and sodium bisulfite feed systems. The new CCT will provide greater hydraulic capacity and increased disinfection efficiency while meeting Class 1 reliability requirements. The use of gas chlorine disinfection and liquid sodium bisulfite dechlorination will be continued, thereby, protecting the environmentally-sensitive receiving waters of the Trask River and Tillamook Bay.

The City currently has a gas chlorine/liquid sodium bisulfate storage, feed, and residual measuring system that will be reused to the extent possible. The new chlorine tank will be built several feet higher than the existing tank to allow for gravity flow of peak flowrates when the Trask River is at an elevated stage. It will have dual basins to meet reliability requirements and to allow either basin can be taken off-line for cleaning and maintenance. The disinfection system will feature automated control of chlorine and sodium bisulfite feeding and provide manual control backup systems.

A high degree of operational flexibility will be incorporated for sampling and control of chemical feeding to facilitate the following goals:

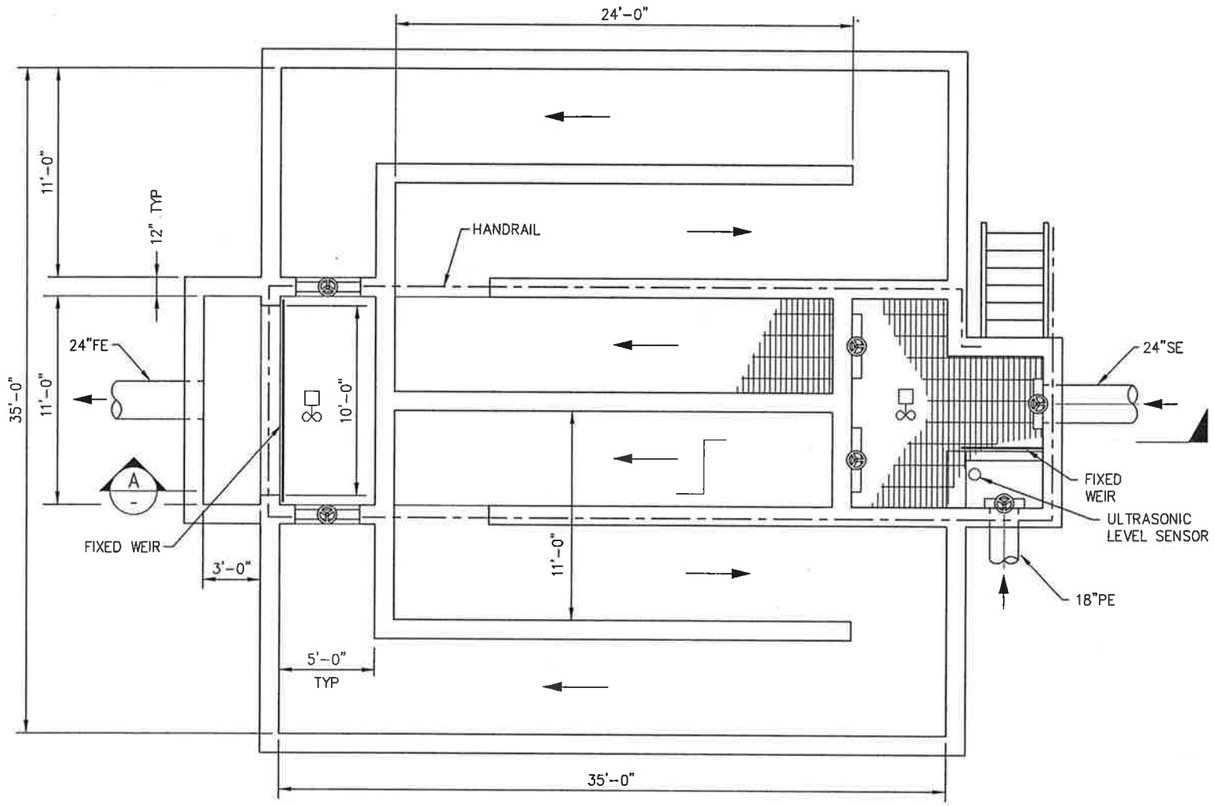
- Optimization of disinfection and dechlorination
- Provide ability to adjust to seasonal wastewater variations
- Provide ability to meet changing wastewater regulatory requirements.

The layout of the new CCT is presented on Figure 8.1.

New Chlorine Contact Tank Layout

Effluent from the secondary clarifier(s) enters the chlorine mixing zone. Chlorine application and mixing will be achieved using a high-speed mixer which draws chlorine gas to the mixing head (by inducing a vacuum) and vigorously mixes the gas with the flow stream. The energy provided by the mixer's impeller will ensure complete and immediate mixing and equal distribution of chlorine between the two basins.

From the chlorine mixing zone, the chlorinated influent flows to both (or either) of the dual contact basins or “channels”. Each channel is approximately 10-feet deep, 5-feet wide, and 80-feet long. The channels provide a serpentine flow path to promote plug flow and eliminate short-circuiting, thus maximizing contact time. Sluice gates will be used to isolate the channels allowing either one to be operated independently while the other channel is drained for cleaning. Both channels can be operated simultaneously under high flow conditions.



PLAN VIEW
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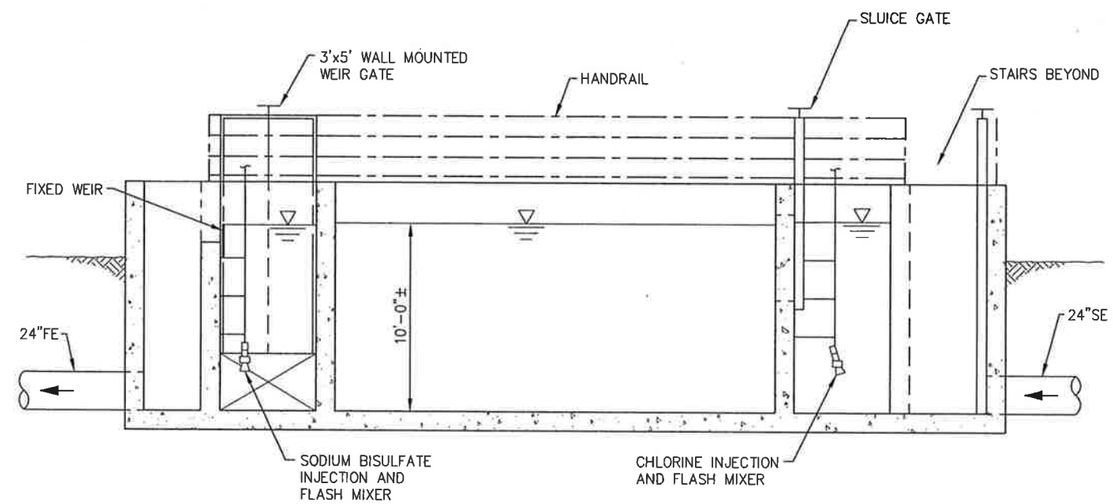

SECTION VIEW
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City of Tillamook, Oregon

CHLORINE CONTACT TANK
PLAN AND SECTION

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Figure 8.1

F:\2004\0491010.20_WWTP_DESIGN\FINAL FREEDESIGN FIG 2006\FIG 8-1 CHLORINE CONTACT TANK.DWG JAN 18, 2006 - 3:21pm

Following chlorine contact, disinfected effluent converges into the dechlorination mixing zone. Using a high-speed mixer, liquid sodium bisulfite will be introduced and rapidly mixed with the flow stream to remove the chlorine residual. This process, referred to as dechlorination, is required since chlorine is toxic to aquatic life and the receiving waters in Tillamook are environmentally-sensitive.

Stairs will be provided to access the CCT. Grating will provide a walkway and access to the flow control gates. Hand railing will be provided as required.

Design Criteria for New Chlorine Contact Tank

The DEQ stipulates that the volume of the CCT must provide a hydraulic residence time of at least 15 minutes at design flow, and, EPA reliability standards for Class I treatment plants require that "there shall be a sufficient number of contact units of a size such that with the largest flow capacity unit out of service, the remaining units shall have a design flow capacity of at least 50% of the total design flow to that unit process". At least two basins are required to meet this requirement.

Historically, peak instantaneous flow (PIF) has been used as the design basis for sizing a contact tank. For this project, the DEQ will allow peak day flow (PDF) to serve as the basis of design. The rationale for this request was as follows:

- The PIF for Tillamook is based on the 5-year storm event and would, therefore, occur very rarely.
- The PIF will occur during a winter month. Any maintenance requiring that one of the dual basins be taken off-line can be done during the summer ensuring that the total contact tank volume is available when peak flow is likely to occur.
- The disinfection system will use a high-speed mixer that greatly increases the efficiency of the contact process and reduces the need for extended contact time. The commonly-used 15-minute retention time criterion assumes that full contact with the disinfectant may take several minutes. This "lag" time can be eliminated with high-speed mixers and, therefore, the contact time required for effective disinfection is reduced.

Based on the peak day flow criterion, the new CCT will provide 15 minutes of contact time at a flowrate of 7 MGD. This translates to a basin volume of approximately 73,000 gallons.

Automated Control of Chemical Feeds

The chlorination/dechlorination process will be automated in order to provide immediate response to changing chlorine demand, thus ensuring adequate disinfection and chemical savings.

The overall objectives of the automated system are: 1) to provide an automated system which applies a chlorine dose adequate to reliably reduce microbiological plate counts to meet regulatory requirements; 2) to minimize chlorine consumption; and 3) to eliminate the chlorine residual while not overfeeding the dechlorination chemical.

Control of Chemical Feeds

Both chlorine and sodium bisulfite control will use a compound loop system integrating flowrate and chemical residual concentration to pace chemical feed rates. The signal from the effluent flow meter, located just downstream of the CCT, will be used to pace chemical delivery in proportion to flowrate. The chlorine dosage will be “trimmed” in response to the chlorine residual or the oxidation reduction potential (ORP) in the upstream reach of the of the CCT channel. Trimming the chlorine dosage to meet changing demands will maintain the required chlorine residual and avoid overfeeding chlorine. The ability to change dosage rates in response to demand becomes especially important during periods of primary effluent bypass (i.e., peak flow bypass) when the chorine demand could be proportionally greater than for secondary effluent alone.

To efficiently react to changing demand, it is normally recommended to sample at a point two to five minutes downstream of the chemical injection point. Thus the sample location should be modified with large changes in flowrate. Control accuracy would be reduced if chlorine residual were instead sampled at the downstream end of the CCT due to the extended lag time and its affect on the action/response time. If a residual chlorine meter is used, a moveable, submersible pump will provide the sample stream. If ORP is chosen for trimming the chlorine dosage, the sample pump could be eliminated as the ORP system uses a submersible probe. A walkway will be provided above the center of the CCT, which will provide access along the length of the two center basins allowing for flexibility in the location of the sample point. Residual chlorine concentration should be nearly identical in either channel so only one sample point is required to serve both channels. The residual chlorine at the downstream end of the CCT should be sufficient to meet permit limits with regard to coliform inactivation but concurrently be minimized to reduce chemical consumption and required dosages of dechlorination chemical.

Sodium bisulfite will be added to the CCT effluent in the downstream mixing zone to remove residual chlorine. Bisulfite delivery will be trimmed in response to residual chlorine or sulfite concentration. The sample point for the dechlorination process will be located downstream of the bisulfite injection point. Sodium bisulfite will be fed to achieve zero chlorine residual. While it is possible to analyze for either residual chlorine or residual sulfite to control dechlorination chemical feed, it is anticipated that the City’s existing chlorine analyzer will be reused for this application.

Recommended Disinfection System

Operational Considerations

The disinfection system should be designed with a high degree of operational flexibility. Wastewater characteristics and regulations will change with time; therefore, operations staff will need to address those changes with modifications in disinfection practice. In addition, flexibility of operation will facilitate the efforts of staff to determine best practice for future conditions, a process that is empirical in nature. Also, if future disinfection regulations become based on concentration x time (as followed in potable water treatment) rather than time alone, the disinfection system recommended below can easily be modified to accommodate this approach.

Disinfection Process Control

In order to monitor the concentration or effectiveness of the chlorine applied at the upstream location, and to “trim” chlorine delivery, either a chlorine residual analyzer or an ORP probe could be utilized. The ORP system has the advantage of being able to rapidly measure the overall oxidizing potential of the wastewater without sample pumps and chemical buffering systems. For many installations, ORP control has resulted in substantial chemical savings while providing highly effective and reliable disinfection. A residual chlorine meter would have the advantage of being familiar to the WWTP staff. The final decision should be based on staff preference.

Dechlorination Process Control

The City owns and operates a Wallace and Teirnan DEOX/2000 dechlorination analyzer. For economic considerations, it is recommended that this analyzer continue to be utilized. Control signals from the unit will be used to adjust or “trim” delivery of the sodium bisulfite feed pump. Because the sampler will receive a sample from a point downstream of dechlorination, this control system is termed a "feed-back" control system.

Manual Backup Control

Both the disinfection and dechlorination feed systems should have provisions for manual operation. The City’s existing manual feed systems will be maintained.

A schematic of the recommended automated control system is provided in Figure 8.2.

Chlorine Storage and Feed

It is anticipated that the chlorine dosage will be between 3 mg/l and 10 mg/l and that the liquid sodium bisulfite dose will be between 3 mg/l to 6 mg/l. The existing gas chlorine feed system should be field evaluated to ascertain if its condition and peak capacity warrant replacement of this equipment. It is recommended that 150-pound cylinders will continue to be used for chlorine

gas storage. Local authorities with regulatory jurisdiction generally prefer the use of 150-pound containers over larger one-ton containers. The need for a “fail-safe” valve and gas detection system should be discussed with the local Fire Marshal in the early stage of design as he will dictate the requirements for safety equipment associated with gas chlorine storage. Since the existing gas chlorine storage system will be maintained, there may not be a requirement for safety upgrades. The existing 500-gallon sodium bisulfate storage tank, building, and feed equipment will be reused.

Table 8.1 below provides an estimate of chlorine feed rates based on dosages between 3 mg/l and 10 mg/l and flowrates of 1 MGD (Q_{ADWF}), 4 MGD (Q_{AMWWF}), and 7 MGD (Q_{PDF}).

Table 8.1: Anticipated Range of Chlorine Feed Rates

	Chlorine Usage, pounds per day		
Cl_2 Dosage (mg/l)	*@ 1 MGD*	*@ 4 MGD*	*@ 7 MGD***
3	25	100	175
6	50	200	350
10	83	333	583

* Potential increased chlorine demand of bypass flow not addressed.

If chlorine gas consumption tends toward the higher range requiring frequent replacement of 150 lb cylinders, a conversion to one-ton cylinders could be considered. The existing storage facility in the Administration/Lab Building is equipped to handle one-ton cylinders but is not currently equipped with the required safety equipment. Fail-safe isolation valves and containment vessels are now an acceptable alternative to gas scrubber systems making the use of one-ton cylinders more attractive from a cost standpoint. From a safety standpoint, the smaller 150-pound cylinders are preferred if the capacity is adequate. Operational experience with the updated treatment process will enable the City to make a more informed decision following startup of the new WWTP. In the interim, the smaller cylinders should be sufficient.

Chemical Delivery to the Application Point

With the addition of the high-speed chemical mixers, chlorine gas can be delivered under vacuum to the application point rather than as a chlorine solution under pressure. This is a preferred approach since it eliminates the possibility of leaking a concentrated solution of chlorine from yard piping and it simplifies the feed system as it eliminates the need for a pressurized carrier-water system. Liquid sodium bisulfite can be pumped directly to the mixer.

Technical Memorandum No. 9

Subject: Yard Piping, Hydraulic Control Structures, and Plant Utilities
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Tim Caire, P.E.

Overview

Yard piping for this project can be categorized according to three general classes. The first is process piping, which serves all main and auxiliary process streams. The second is plant utility service water; this consists of non-potable water and potable water systems. The third is plant raw sewage influent piping. This Technical Memorandum addresses each of these three classes.

Yard Piping

There are numerous components which must be defined in order to produce a complete piping system. Choices for these components have been based on the specific process application, working pressure, and physical environment in order to design piping systems which meet the requirements of the project. These components are presented and discussed below.

A. Process Piping

1. Definition of Pipe, Fittings, and Appurtenances

- Material: Ductile Iron (AWWA C151)
- Pressure Class: Minimum 150 psi
- Lining:
  - *Thickened sludge; scum*: Glass or Ceramic Epoxy
  - *All other piping*: Cement Mortar
- Gaskets: SBR
- Bolts and Tie Rods:
  - Exposed: Electroplated Zinc or Cadmium Steel
  - Submerged: Type 304 Stainless Steel
- Buried Pipe - Additional Requirements:
  - Wrap (buried): Polyethylene Encasement
  - Minimum Depth of Cover: 3 feet
  - Flexible Couplings: 2 adjacent to all structures; restrained

2. Definition of Pipe Joints

Pipe joints will be either mechanical joint (MJ), push-on joint (POJ), or flanged (FL) according to the table below. Restraint will be provided on all MJs and POs in pressurized piping. All buried fittings will be restrained.

Table 9.1 – Pipe Joints

	Pipe, Buried	Fittings, Buried	Pipe & Fittings, Exposed
Pressurized	POJ, restrained per Spec Section 15050	MJ, restrained	FL
Gravity Flow	POJ, not restrained	MJ, restrained	FL

3. Wall Penetrations

Wall penetrations will be constructed according to the table below.

Table 9.2 – Wall Penetrations

	Through New Structures	Through Existing Structures
Buried, not submerged	CIS	LS
Buried, submerged	CIS	CIS
Exposed	LS	LS

CIS = cast iron sleeve (sleeve cast-in-place for new structure or grouted into cored penetration of exist structure). End fittings (plain, flanged, or MJ) per Table shown in Section 2 above.
LS = link seal (link seal installed in PVC sleeve which is cast into new wall or grouted into core drill of exist structure).
Note: CIS can be substituted for LS in any instance.

B. Plant Utility Piping

Utility piping, which will serve the nonpotable water and utility water systems, will be constructed of PVC for buried installations and of copper for exposed installations. The PVC will be Schedule 80 with solvent-welded joints, and the copper will conform to ASTM B88, Type L, with soldered joints.

C. Raw Sewage Piping

New raw sewage gravity piping will be constructed of PVC Sewer Pipe, in conformance with ASTM F679. Joints will be push-on with gaskets; gasket material will be SBR. Direct connections to existing RCP sewage piping will not be required, as manholes will receive both pipelines.

D. Outfall Piping

Outfall pipe shall be reinforced concrete pipe.

Hydraulic Control Structures

Two new structures will be constructed for regulation and routing of process flow streams. Both structures will work in conjunction with the aeration basins. One or both of these structures may be constructed integral to the basin. From a hydraulic perspective, the new Primary Effluent Flow Control Structure (PEFCS) will be immediately upstream of the aeration basins and provide a means to bypass a portion of the flow around the secondary treatment system under peak flow conditions. The Mixed Liquor Flow Splitter Box (MLFSB) will be immediately downstream of the aeration basins, and will be physically attached to the structure. The MLFSB will receive the mixed liquor flow stream (i.e. the aeration basin effluent) and divide it evenly between the secondary clarifiers. In addition, the two control structures will be designed to facilitate a future expansion. The PEFCS and MLFSB are described in greater detail below.

1. Primary Effluent Flow Control Structure

The PEFCS will receive up to 10 MGD of effluent from the existing primary clarifiers, and will also receive up to 5 MGD of RAS from the new secondary clarifiers. These streams will combine and then enter a 24-inch pipeline en route to the aeration basin; however, prior to combining the primary effluent and RAS streams, the PEFCS will divert any primary clarifier effluent in excess of 5 MGD. The diverted stream will be routed to the chlorine contact tank, bypassing the aeration basins and secondary treatment.

A more detailed description of the PEFCS follows. Primary effluent will enter the PEFCS and then flow through a cutthroat flume en route to the aeration basins, or will be diverted to the bypass system as discussed. An electrically-actuated modulating weir gate will be used to control bypass flow so as to maintain a setpoint maximum flow rate of 5 MGD to the secondary process. The motorized weir gate will be controlled using a signal from an ultrasonic level measurement instrument, which will relate water level in the flume to flowrate. Space will be provided for future addition of a second flume; when two flumes are in service, the PEFCS will split flow between two aeration basins.

The point of entry for RAS will be an inlet box located at the opposite end of the structure from entry of PE. RAS will flow through a cutthroat flume (to split flow) prior to dropping into a box which will also receive PE. Space will be provided for future addition of a second RAS flume. In addition, the RAS inlet box will contain a bypass pipe which can route RAS directly to the aeration basin(s), without mixing with PE. The RAS bypass pipeline would be utilized when operating the aeration basins in the "sludge re-aeration" mode, discussed in Technical Memo No. 5.

Stop logs or gates will be provided for isolation of all cutthroat flumes. The PEFCS will be covered with walkway grating. A handrailing will be provided as the structure will be elevated above grade. The PEFCS is illustrated on Figure 9.1. It may be logical to combine this structure with the aeration basin. This determination will be made during final design.

2. Mixed Liquor Flow Splitter Box

The MLFSB will receive mixed liquor from the aeration basins. The structure will contain two cutthroat flumes, which will effectively divide the mixed liquor stream evenly among the secondary clarifiers. Space will be provided for addition of a third flume, to serve a future third clarifier. The flumes will be sized to provide up to 5 MGD capacity. Stop gates in the influent channel will be installed for basin isolation. The MLFSB is shown on Figure 5.1.

Plant Utilities

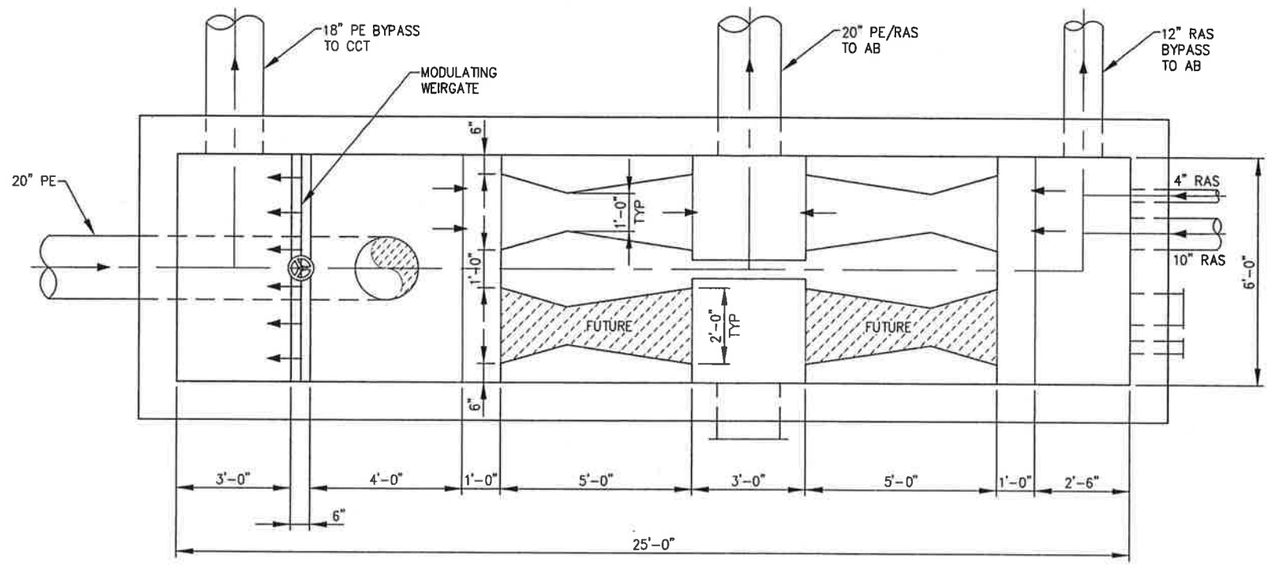
Currently, the WWTP does not have a non-potable water (NPW) system; all plant water is provided by the City of Tillamook. The WWTP upgrade will include a new NPW system and, for potable service, the existing connection to the City's water system will be maintained. The new NPW system will provide disinfected wastewater effluent (from the chlorine contact tank) to the plant, and will serve hose bibs and a variety of process equipment including basin foam sprayers, the WAS thickener belt, the septage receiving station, and possibly for irrigation of landscaping. The use of reclaimed water for non-potable demand is beneficial in that it reduces the need for City-supplied drinking water for these purposes. Potable water will continue to be provided for drinking water fountains, eye washes, chemical feed systems, and the existing laboratory building.

Typically, water intended for reuse outside a WWTP is classified by DEQ as reclaimed water and must meet strict guidelines. Since the NPW at Tillamook will be treated and will be used exclusively on-site, it is exempt from DEQ guidelines. The rules governing this exemption are dictated by the Oregon Administrative Rules (OAR 340-55-0013) and are as follows:

1. The reclaimed water that is used is disinfected, oxidized wastewater; and

2. Reclaimed water that is used for landscape irrigation shall be confined to the treatment plant site. No spray or drift shall be allowed off the treatment plant site. The treatment plant site shall not include property that is not contiguous to the parcel of land upon which the treatment plant is located.

The new NPW system will deliver water of sufficiently-high quality for these applications. Please refer to Technical Memos Nos. 5 and 8 for a description of the biological treatment and disinfection processes associated with the reclaimed NPW. Reclaimed water also will pass through an inline strainer prior to entering the NPW system.



PLAN VIEW–TOP
SCALE: 3/8"=1'-0"

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City of Tillamook, Oregon

Primary Effluent Flow
Control Structure (PEFCS)
Top Plan

0491010.20

Figure 9.1

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The system will be designed to meet all demands concurrently. Estimated NPW demands are shown in Table 9.3 below. Total annual usage is approximately 5M gallons and the maximum instantaneous demand is assumed to be about 110 gpm.

Table 9.3: Non-Potable Water System Demands

NPW Demand	Instantaneous Demand (gpm)	Annual Usage (gallons)
Fine Screen Washer	20	525,600 [a]
Gravity Belt Thickener	30	1,728,000 [b]
Onsite Irrigation	20	216,000 [c]
Plant Washdown	20	438,000 [d]
Septage Receiving Station	20	62,400 [e]
Total	**110**	**2,970,000**

Notes:
(a) Assumes approximate use of 5% per day, 365 days per year.
(b) Assumes approximate use of 6 hours per day, 160 days per year.
(c) Assumes approximate use of 2 hours per day, 90 days per year
(d) Assumes approximate use of 1 hour per day, 365 days per year.
(e) Assumes approximate use of 1 hour per day, 52 days per year.

The NPW system will utilize a pump variable frequency drive controller. This unit is an economical alternative to standard hydropneumatic tanks for maintaining a steady system pressure in spite of fluctuations in water demand (a small pressure diaphragm tank is required). The NPW system will be installed in the Blower and Standby Generator Building.

Technical Memorandum No. 10

Subject: Outfall Improvements and Permitting Requirements
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Tim Caire, P.E., Harry Ritter, P.E., and Gregg Bryden, CHMM

Outfall Improvements

Kennedy/Jenks Consultants completed a mixing zone analysis in 2003 for the existing WWTP and discharge structure. This study, entitled "Wastewater Treatment Plant Effluent Water Quality Study" (Effluent Study), was reviewed by the Oregon Department of Environmental Quality (DEQ). The DEQ was in general agreement with the Effluent Study; a memorandum was issued by the DEQ on 27 August 2004 which discusses their understanding of the study and required outfall improvements, along with related suggestions. A copy of this memorandum is provided following this document.

The goal of the Effluent Study and of DEQ guidance is to ensure that outfall dilutions of WWTP effluent are adequate to ensure compliance with current and future requirements of Tillamook's NPDES Effluent Discharge Permit.

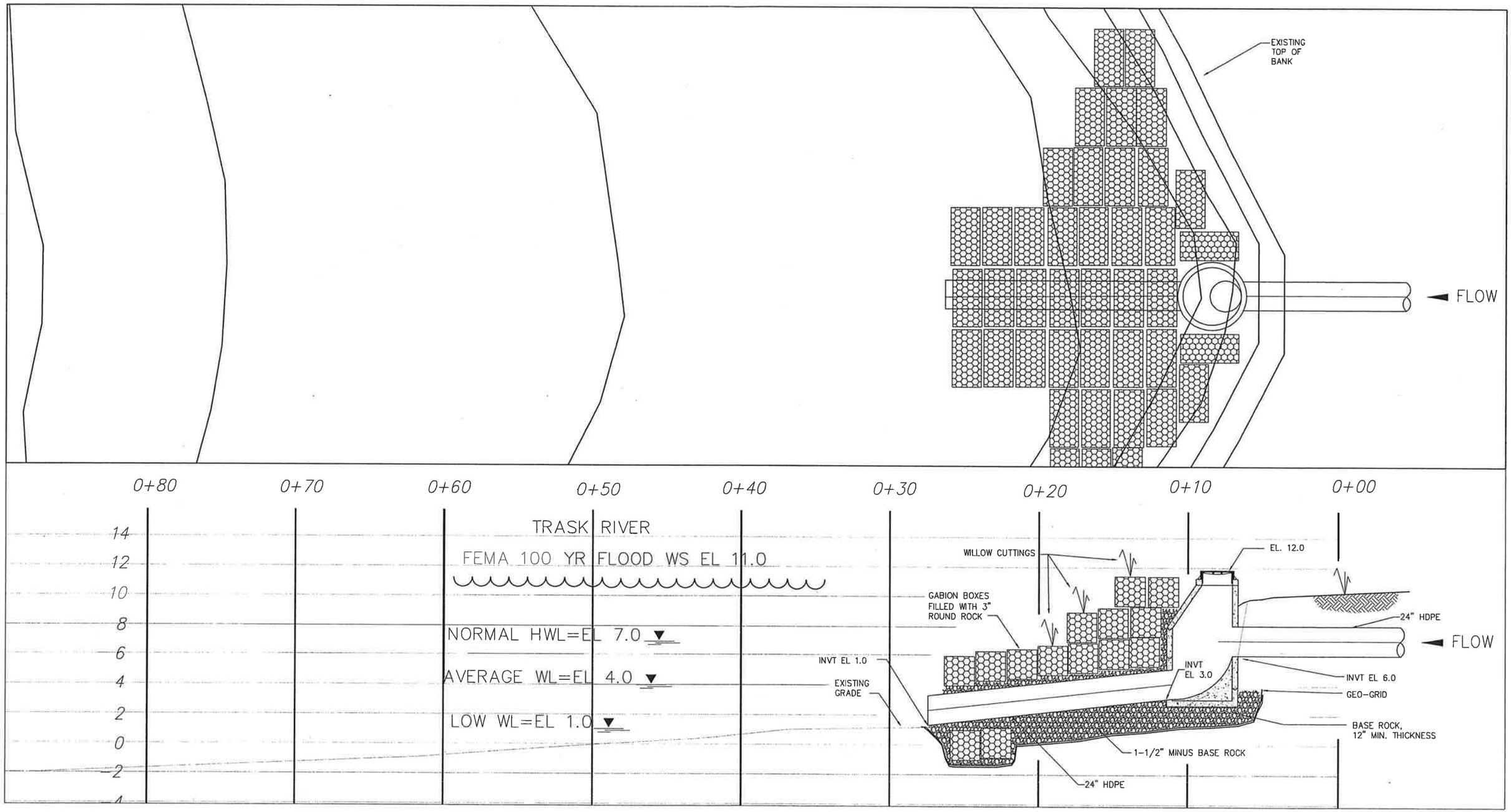
The Effluent Study concluded that mixing would be improved if the outfall were lowered to the river bottom, and extended a short distance into the mixing zone Both the Effluent Study and the DEQ estimate that dilution at the edge of the mixing zone would be improved from a value of approximately six to a value of approximately 19 during ADWF conditions by extending the outfall by 10 feet and also lowering it approximately five feet such that it contacts the river bed.

The existing outfall will be reconstructed so that these improvements to the mixing zone are realized. The design of these improvements will be undertaken early during the final design phase. Upon completion of this design component, a permit application will be submitted to the Oregon Department of State Lands (DSL). A new outfall, as described above, will be constructed adjacent to the existing outfall. Once finished, it will be placed in service and the existing outfall will be decommissioned. Figure 10.1, Effluent Outfall Structure, shows the preliminary concept for the modified outfall.

It should be noted that concerns over chronic ammonia toxicity will be mitigated not only due to increased effluent dilution, but also due to nitrification which will be carried out within the aeration basins, as discussed in Technical Memo No. 5. As noted in the Effluent Study, effluent temperature should not require mitigation. However, standards for temperature and toxic constituents will probably become more stringent in the future. The DEQ will also implement a policy for Reasonable Potential Analysis which may increase monitoring requirements.

Permit Requirements

Modification of the effluent outfall will require a joint permit from US Army Corps of Engineers (USACE) and the Oregon Department of State Lands (DSL). USACE is the federal agency responsible for enforcement of the Rivers and Harbors Act and the Clean Water Act, Section



Kennedy/Jenks Consultants

City of Tillamook, Oregon

EFFLUENT OUTFALL STRUCTURE

0491010.20

Figure 10.1

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404. DSL imposes state removal/fill laws when over 50 cubic yards of soil are involved. The joint application requires detailed site and project information, including habitat information (in the form of a brief Biological Assessment).

Known factors and assumptions which may influence the permit process are:

- A temporary coffer dam may be needed for the construction
- More than 50 cubic yards of fill material (bank armoring or concrete structure) may be needed to protect the outfall
- There is no salmon spawning habitat in the area (based on a 2000 letter from ODF&W)
- This assessment does not include other non-environmental permits such as local building and grading permits.

Beginning 1 January 2006, DSL may be able to authorize this work via a USACE Statewide Programmatic General Permit (SPGP). If so, at the time of application DSL will take the lead role for processing the permit application.

A SPGP is a type of general permit, issued by USACE that authorizes certain activities that also are regulated by another level of government. This SPGP is based on the existing regulatory program administered by DSL under the authority of the state Removal-Fill Law. The SPGP process is intended to reduce duplicative state and federal regulatory processes, simplify the application process for applicants, and make better use of limited government resources.

Additional information on the SPGP can be found at:
http://www.oregon.gov/DSL/news/pr0543_spgp_signed.shtml

Information on the Oregon Removal-Fill Program can be found at:
http://www.oregon.gov/DSL/PERMITS/r-fintro.shtml

The work can be conducted under two Nationwide General Permits which, unlike individual permits, do not require public hearings. Nationwide Permit No. 7 will cover pipeline construction. Nationwide Permit No. 13 will cover placement of fill material for bank protection. The work would also be conducted under a DSL general authorization. DSL may also require an easement for the pipe, if the City does not already have one. The easement application form can be found at: http://www.oregon.gov/DSL/LW/docs/easementapp.pdf.

USACE will consult with NOAA Fisheries and US Fish and Wildlife Service about Endangered Species Act issues. Supporting documentation, including the brief Biological Assessment, will facilitate this consultation. The Oregon Department of Fish and Wildlife also will be consulted regarding in-water work, potential impacts to habitat, and scheduling.

USACE will also discuss water quality certification requirements with DEQ. Nationwide Permit No. 7 is pre-certified; however, Nationwide Permit No. 13 is only partially-certified and will require coordination by USACE with DEQ. Additional information on the current USACE permit process can be found at: https://www.nwp.usace.army.mil/op/g/process.asp.

It is anticipated that the permit application process will require approximately one year. For this reason, the outfall design and permit application will be completed early during the final design phase of the plant expansion project, as discussed above. Also, construction of outfall improvements could be scheduled for the end of the overall project, if additional time is required for permitting activities.

Technical Memorandum No. 11

Subject: Waste Activated Sludge Thickening
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Harry Ritter, P.E.

Introduction

The first purpose of this technical memorandum is to evaluate alternate technologies for thickening waste activated sludge (WAS) prior to anaerobic digestion. Options for this unit process were not developed in the Facility Plan. This memorandum will investigate possible options and recommend the best alternative based on capital and O&M costs, performance, and ease of operation.

The second purpose of this memorandum is to state the design criteria for the selected thickening technology and discuss design details related to the thickening application.

Background

The proposed WWTP expansion will incorporate the existing primary clarifier, an activated sludge biological process with clarification, and anaerobic digestion. WAS from the biological process will be thickened prior to digestion due to its dilute nature. Primary sludge will not be thickened beyond what is achievable in the clarifier although provisions will be incorporated in the sludge piping so that it can be easily incorporated in the future if required. Digested sludge will be stored onsite until land applied as a dilute liquid.

Sludge Production

Table 11.1 shows the predicted startup and future sludge production values prior to digestion. The values shown in this table assume that 30% of the BOD and 60% of the TSS is removed in the primary clarifier and that the overall yield coefficient is 0.7 lb MLSS/lb BOD.

Table 11.1 – Sludge Production for the Tillamook WWTP

Item	Startup		Design	
	Average Month	Peak Month	Average Month	Peak Month
Raw Influent BOD (ppd)	1,250	1,485	2,100	2,475
Raw Influent TSS (ppd)	1,250	1,485	2,100	2,475
Primary Effluent BOD (ppd)	875	1,040	1,470	1,733
Primary Effluent TSS (ppd)	500	594	840	990
Primary Solids Concentration (%)	3.5	3.5	3.5	3.5
Primary Solids Volume (gpd)	2,570	3,050	4,230	4,990
Primary Solids VSS (%)	70	70	70	70
Primary Solids TSS (ppd)	750	891	1,260	1,485

Item	Startup		Design	
	Average Month	Peak Month	Average Month	Peak Month
Primary Solids VSS (ppd)	525	624	882	1,040
WAS Solids TSS (ppd)	613	728	1,029	1,213
Average WAS Concentration (%)	0.8	0.8	0.8	0.8
WAS Volume (gpd)	9,200	10,900	15,420	18,180
WAS TSS (ppd)	613	728	1,029	1,213
WAS VSS (%)	80	80	80	80
WAS VSS (ppd)	490	580	820	970

Table 11.2 shows the daily volume of un-thickened and thickened WAS at various concentrations.

Table 11.2 – WAS Volume at Various Concentrations Following Thickening (gpd)

Item	Startup		Design	
	Average Month	Peak Month	Average Month	Peak Month
WAS Volume @ 0.8% (a)	9,200	10,900	15,420	18,180
WAS Volume @ 1.5% (b)	4,900	5,800	8,200	9,700
WAS Volume @ 3.0% (c)	2,400	2,900	4,100	4,800
WAS Volume @ 6.0% (d)	1,200	1,500	2,100	2,400

(a) Un-thickened WAS
(b) Lower end of thickened WAS range
(c) Middle value for thickened WAS
(d) Upper end of thickened WAS range

Table 11.2 indicates that significant WAS volume reduction is possible by thickening prior to digestion. This is essential in order to minimize digester volume and to reduce the energy required to heat the incoming material to the required temperature.

Thickening Alternates under Consideration

The following sludge thickening options are considered:

1. Gravity Thickening
2. Gravity Belt Thickening
3. Rotary Drum Thickening
4. Dissolved Air Thickening
5. Centrifugal Thickening

The alternatives are discussed in detail below:

Gravity Thickening

Gravity thickening is a process that uses a device similar in construction to a conventional secondary clarifier. A sloped-bottom basin of concrete or steel construction is equipped with an influent baffle, a rotating rake arm with scrapper blades and a bridge-mounted drive, and an effluent weir mounted to the perimeter of the basin. Sludge is introduced into the thickener through the influent baffle and allowed to settle. Concentrated sludge is moved to a hopper by the rotating rake arm where it can be pumped from the tank. Floating material is removed through a scum box. Supernatant flows over the perimeter weir and is returned to the plant recycle stream.

Thickened sludge concentrations of 1.5% to 5% are possible depending on the nature of the sludge being processed but normally average less than 2% for WAS alone. Polymer or other chemicals can be used to enhance performance. Gravity thickeners can be used to process either primary or secondary sludge or a mixture of the two. The key to efficient operation of a gravity thickener is to introduce dilute sludge to the device. Gravity thickening of primary sludge can be achieved with excellent results but odor generation and corrosive gas production must be managed. Gravity thickening of WAS is enhanced when mixed with primary sludge. WAS alone will not normally settle effectively in a gravity thickener due to its large surface area per unit mass. For this reason gravity thickeners are seldom used for WAS thickening alone.

A gravity thickener is the lowest capital cost option considered. When co-thickening WAS and primary sludge, odors must be contained and treated. Table 11.3 shows the advantages and disadvantages of Gravity Thickening:

Table 11.3 – Advantages and Disadvantages of Gravity Thickening

Advantages	Disadvantages
lowest capital cost option	poor performance for WAS alone (1% to 2% TS)
lowest operating cost	supernatant can be hard to treat
building not required	odors must be contained and treated
easy to operate	lower overall solids concentration than other options
	scum/grease accumulation can create maintenance chore
	corrosion of metal and concrete can be a problem

The preliminary cost estimate for an appropriately sized gravity thickener with a concrete basin and a stainless steel mechanism and odor control is $150,000.

Gravity Belt Thickening

A gravity belt thickener (GBT) consists of a mechanical device utilizing a porous fabric belt moved over rollers by a variable-speed motor. The sludge is first conditioned by polymer then

fed to a distribution box at one end of the machine. The box is used to distribute the sludge across the width of the moving belt. As the sludge is moved alone the length of the belt, it is ridged and furrowed by a series of multiple plow blades. Free water (filtrate) is released from the sludge and falls through the belt. Thickened sludge is dropped off the end of the belt, collected into a hopper and then pumped to the digester. The belt is continuously washed by a stream of high pressure water. The filtrate and wash water are recirculated back to the treatment process.

The GBT has been used to effectively thicken both primary sludge and WAS. It can also co-thicken both types of sludge simultaneously. Thickening of primary sludge will produce strong odors. If odor control is required, the GBT must be installed inside a building. Odor control is usually accomplished using a fume hood mounted over (or under) the machine. Plastic curtains surrounding the machine can be used to help contain the odor. The GBT is easy to operate, can be left unattended for extended periods, and can be fitted with alarms to automatically shut down if a problem occurs. The GBT is a relatively large piece of equipment compared to other mechanical options and requires a sump under the machine to trap filtrate and washwater as it falls through the belt. This sump is normally integrated into the concrete slab increasing the building cost. The GBT can thicken WAS to a concentration of 4% to 6% depending on the nature of the sludge and the amount of polymer used. A WAS batch tank can be used to store WA prior to processing but the GBT is often fed directly from the primary or secondary clarifier.

A GBT is an intermediate capital cost option for sludge thickening. For primary sludge applications, odors must be contained using a specially designed fume hood. The fume hood also serves to remove moist and corrosive vapor produced by the thickened sludge. It is not recommended to install a GBT in a building without vapor control due to corrosion considerations. Table 11.4 shows the advantages and disadvantages of the GBT:

Table 11.4 – Advantages and Disadvantages of the GBT

Advantages	Disadvantages
good thickening performance (4% to 6% TS)	requires polymer addition
can thicken primary and WAS	relatively large footprint
low power consumption	requires a building for odor control and weather protection
can operate unattended	odors control requires a large fume hood and appurtenances
low maintenance requirements	Intermediate capital cost
	requires high pressure belt wash stream

The preliminary cost estimate for an appropriately sized gravity belt thickener with the appropriately sized building and odor control is $345,000.

Rotary Drum Thickening

A rotary drum thickener consists of a perforated stainless steel drum that rotates within a stainless steel or FRP enclosure and a conditioning tank that flocculates the sludge through the addition of polymer. The flocculated sludge is introduced into the rotating screen. Free water (filtrate) falls through the perforations in the screen. Thickened solids are conveyed along the length of the screen by an auger and deposited at the front of the machine where it is collected and pumped away. A high pressure water spray is required to continuously wash the screen. The wash water and the filtrate is captured under the machine and directed to the plant recycle stream.

The rotary drum thickener is a simple and reliable device and has a solids capture rate of 90% to 95%. Solids can typically be thickened to 4.5% with reasonable polymer consumption (8 lb. to 12 lb. per dry ton of solids). The machine is partially enclosed, reducing the omission of odors and can be installed indoors or outdoors. Power consumption is relatively low, about 7.5 hp for the size machine under consideration. A rotary drum thickener can be run with minimal operator attention once the proper polymer dosage is selected for a given sludge type. Sludge can be pumped directly from the clarifier to the machine or from a WAS batch tank. The machine has a small footprint, about 33% required for a gravity belt thickener, allowing it to be installed in a substantially smaller building.

The rotary drum thickener is the lowest capital cost mechanical alternative considered. Due to its smaller footprint, building costs are reduced when compared to the GBT. Odors are easier to contain than for other options since the unit is partially enclosed, reducing the cost of odor containment. Table 11.5 shows the advantages and disadvantages of the rotary drum thickener:

Table 11.5 – Advantages and Disadvantages of the Rotary Drum Thickener

Advantages	Disadvantages
intermediate thickening performance (up to 4.5%TS)	slightly higher polymer consumption than GBT
can thicken primary and WAS	process is not observable (enclosed)
low power consumption	Slightly lower capture rate than GBT
lower vapor production than GBT	requires high pressure drum washing stream
odors easier to contain than GBT	
lowest capital cost mechanical device	
can operate unattended	
small footprint	
low maintenance requirements	

The preliminary cost estimate for a rotary drum thickener with the appropriately sized building and appurtenances is $240,000.

Dissolved Air Flotation Thickening

A dissolved air floatation (DAF) thickener system consists of a clarification tank with a collection device and an air pressurization unit. The tank can be either rectangular or circular. The collection system is similar in design to a conventional secondary clarifier except that the skimmer units are larger and there may be up to eight or more per basin. Pressurized air is dissolved into the liquid stream in an air saturation tank. The pressurized solution is then released through a back pressure valve into the tank causing minute air bubbles to form. The majority of solids rise to the water surface with the aid of the air bubbles. As the solids collect on the water surface, they are collected by the skimmer arms and dropped into the float trough. Clarified supernatant falls over the discharge weir and is handled as a plant recycle stream. Heavier solids that will not float, sink to the bottom and are collected in a sump. The heavier solids primarily consist of inert material and grit and can be washed in a cyclone separator and then disposed. Polymer is sometimes used to help flocculate solids making them more amenable to DAF thickening. A DAF system is more complex than the other mechanical means of mechanical thickening discussed herein as pressurization pumps, an air compressor, and a saturation tank is required in addition to the mechanical collection system installed in the tank. However, all of this equipment is relatively straightforward and can be kept reliable with routine maintenance.

Historically, DAF thickening has been used to thicken WAS as these solids have a specific gravity close to water, making them float easily with the aid of the air bubbles. Primary sludge can be thickened in a DAF unit but since this type of sludge has a higher specific gravity, it is not usually a good option. DAF units can be covered for odor control. Installing the tank and appurtenances indoors would be recommended in Tillamook to protect them from the salt air.

The DAF thickener is an intermediate capital cost alternative for sludge thickening. The unit sized for the Tillamook facility would be 8-feet in diameter and have a side water depth of 6.5-feet. This unit could be skid-mounted and constructed of coated steel. Table 11.6 shows the advantages and disadvantages of a DAF thickener:

Table 11.6 – Advantages and Disadvantages of the DAF Thickener

Advantages	Disadvantages
good WAS thickening performance	Not a good choice for primary sludge thickening
sludge is aerated and kept fresh	odors/vapors must be contained and treated if primary sludge is thickened
relatively low capital cost	more complex mechanically than other systems
can operate unattended	relatively large building footprint to house DAF plus ancillary equipment
	intermediate maintenance requirements

The preliminary cost estimate for an appropriately sized DAF unit, building and appurtenances is $255,000.

Centrifuge Thickening

A centrifuge thickening system requires a centrifuge, a feed pump, a flow meter, a polymer feed system, a thickened sludge pump, and a programmable logic control (PLC) panel. The centrifuge is a relatively complex machine that rotates at high speed to provide centrifugal force for solids separation. The unit is totally enclosed which partially eliminates the need for odor control. However, there may be a need to contain odors once the sludge has been discharged from the centrifuge. Both primary and secondary sludge can be processed. Polymer addition is necessary. Once the centrifuge has been properly set to thicken a particular sludge, it can be left unattended except for periodic checks. The PLC panel can be programmed to shut down the machine if a problem occurs. The footprint of the unit is relatively small compared to other options. The centrifuge requires significant power and polymer for operation. Power consumption is far greater than for any other option considered. All but routine maintenance is usually performed by a factory-trained service provider due to the complexity of the mechanical systems.

The centrifuge thickener can produce a thicker end product than the other alternatives considered. The unit sized for the Tillamook facility would be approximately 10-feet in length and 3-feet. Additional space is required for the control panel and polymer feed appurtenances. The entire unit can be skid-mounted on a unit approximately 15-feet x 10-feet.

The centrifuge has polymer usage rates that are slightly higher than the other mechanical thickening methods described. However, a centrifuge sized for a thickening application at Tillamook would need to be equipped with a motor in the 30-hp to 40-hp range making it the highest operating cost alternative under consideration. In addition, the capital cost for the centrifuge is the highest of all of the alternatives considered. Additionally, a centrifuge requires specialized service personal that normally must travel considerable distance to the plant site.

Table 11.7 shows the advantages and disadvantages of a centrifuge thickener:

Table 11.7 – Advantages and Disadvantages of the Centrifuge Thickener

Advantages	Disadvantages
best thickening performance	high energy costs
small footprint	complexity of equipment requires specialized repair and maintenance personnel
totally contained (reduced odors)	highest polymer requirements
can thicken primary and secondary sludge	highest capital costs

The capital cost for a small centrifuge, building and appurtenances suitable for Tillamook was estimated at $450,000. Operation costs for a centrifuge would typically be four to eight times higher than for the other options. While a centrifuge offers advantages for larger operations where it can be utilized on a nearly full-time basis, its high capital costs and high O&M costs make it unsuitable for most small treatment plants.

Discussion

Selection of the most appropriate technology is subject to many factors and the final decision can hinge solely on personal preference. The selected technology may be based on proximity of repair personnel, the willingness to trade higher capital cost for lower long-term maintenance costs, the experience of a neighboring treatment plant, or advice from a trusted source. To help facilitate a decision, a matrix was developed to help quantify the advantages and disadvantages of the options presented. Even with this commonly used method, the criteria applied to rate the alternatives are subject to the rater's bias and opinions. However, the matrix can be a useful decision making tool. The following criteria were considered when ranking the alternatives:

- Capital Cost
- Operation and Maintenance (O&M) Cost
- Thickening Performance (ability to meet the concentration objective of 6% WAS)
- Reliability (ability to reliability thicken sludge of variable consistency and nature)
- Ease of Operation (level of operator attention required)
- Dual Service (ability thicken both primary and secondary sludge)

The matrix was developed by first assigning an "importance score" (1 through 5) to each of the six criteria and then assigning a "criteria score" to each of the alternatives based on its ability to satisfy the criteria. The importance score is then multiplied by the criteria score to arrive at a "composite score". The composite scores for each alternative are summed to define the "total score" for the alternative. For example, if Capital Cost is considered very important, it would be assigned an importance score of 5. Since the centrifuge is by far the most capital cost intensive alternative, it would be assigned criteria score of 1. The composite score would be 5 x 1 = 5. On the other hand, since the gravity thickener is the lowest capital cost alternative, it would receive a criteria score of 5 and a composite score of 5 x 5 = 25. Table 11.8 shows the results of the matrix ranking.

Table 11.8 - WAS Thickener Comparison Matrix

Criteria	Capital Cost	O&M	Performance	Reliability	Dual Service	Ease of Operation	
Importance Score (a)	5	3	5	5	3	4	
Option	**Criteria Score for each Option (b)**						
Gravity Thickener	5	5	1	3	1	5	
GBT	3	4	5	4	4	4	
Rotary Thickener	4	4	3	4	4	4	
DAF	4	4	3	4	1	3	
Centrifuge	1	1	5	5	5	4	**Total**
Option	**Composite Score for Each Option (c)**						**Score**
Gravity Thickener	25	15	5	15	3	20	**83**
GBT	15	12	25	20	12	16	**100**
Rotary Thickener	20	12	15	20	12	16	**95**
DAF	20	12	15	20	3	12	**82**
Centrifuge	5	3	25	25	15	16	**89**

(a) Importance Score is the perceived importance of each of the criteria to the Owner and Operating Staff.

(b) Criteria Score is the perceived ability of the process alternative to meet the six criteria.

(c) Composite Score is the product of the Importance Score times the Criteria Score.

Thickener Recommendation

Based on the outcome of the matrix, the GBT would be recommended as the most suitable alternative for Tillamook. The high capital and O&M costs make the centrifuge a poor choice. The inability of the gravity thickener to effectively thicken WAS makes it a poor choice. The DAF does not have the ability to thicken primary sludge and is also likely to require more periodic maintenance due to the many pieces of ancillary equipment required for its operation. It may not be as reliable as the rotary drum and GBT from an operation standpoint. The rotary drum thickener would be a good choice except that it will not reliably concentrate sludge above 4.5% (some manufacturers reject this limit stating that up to 6% is achievable). Based on the considerations made in Technical Memorandum 12 regarding anaerobic digestion, the WAS thickening device must be capable of concentrating to 6% solids.

Thickener Installation

The WAS thickener will be installed in its own room in the Solids Processing Building. The Solids Processing Building is discussed in Technical Memorandum 13. The WAS thickening room will be equipped with a ventilation system to remove excess moisture during operation. The GBT will be equipped with a hood for this purpose. The floor of the room will incorporate a

curb and drain system to catch process drainage from the GBT. All building materials and equipment in the room should be appropriate for a damp, corrosive environment.

To save capital costs, it is proposed that the GBT be fed directly from the WAS pumps. To do this, WAS will be wasted over a 6-hour period, five to six days per week. This will require WAS being retained in the aeration basin for 18-hour period between the wasting cycle but since the aeration basin is designed with an overall SRT of 10-days, this should not present an operational problem. At a minimum, the GBT should be sized to adequately process one day's WAS in a 6-hour period. During design, consideration should be given to adding a day tank in the future for batch storage of WAS. Batch storage would allow for constant wasting of WAS and enable WAS thickening to be done at a higher flow rate (i.e., shorter period). A batch tank would require coarse bubble aeration to facilitate mixing and to keep the sludge fresh and should be sized to store up to two to three days of WAS. The required batch tank volume would be approximately 15,000 gallons per day of storage.

A pump will be required to transfer thickened sludge from the GBT discharge hopper to the digester. A mechanical diaphragm, progressing cavity, or rotary lobe pump can be used for this application. A pump that has the ability to "run dry" without damage would safeguard the pump and simplifies the controls. An emulsion polymer feed system using a 55-gallon drum supply will be used for this project. The polymer feeder will require a source of potable water for dilution. The polymer feeder should be capable of feeding proportional to WAS flow. A flow meter on the WAS line will be used to provide a 4-20mA signal to the polymer feeder.

Design Criteria

The design criteria related to the WAS thickener are listed on Table 11.9.

Table 11.9 – WAS Thickener Design Criteria

Criteria	Unit	Value
Feed Concentration	%	0.8
Required Discharge Concentration	%	6.0
Operation Sessions Per Week	days	6
Daily Hours of Operation	hours	4 to 6
Max Month WAS Loading	ppd	1,213
Max Month WAS Volume	gpd	18,180
Maximum Solids Loading Rate	pph	350 to 235
Maximum Hydraulic Loading Rate	gpm	88 to 60
Average Month WAS Loading	ppd	1,029
Average Month WAS Volume	gpd	15,420
Average Solids Loading Rate	pph	300 to 200
Average Hydraulic Loading Rate	gpm	75 to 50
Average to Peak Thickened Sludge Discharge	gpm	5 to 15

Based on the requirements shown in Table 11.9, the maximum hydraulic loading rate of 88 gpm would correspond to processing the Maximum Month Loading in 24-hours per week (6 days at 4 hours per day). The lowest design flow rate would correspond to processing the Average Month Loading in 36-hours per week (6 days at 6 hours per day). Modern gravity belt thickeners are normally rated to process 100 gpm to 150 gpm per meter of belt width. Therefore, a 0.75 meter or 1.0 meter machine would be appropriate.

Technical Memorandum No. 12

Subject: Anaerobic Digestion and Sludge Storage
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Harry Ritter, P.E. and Monty Hazlehurst, P.E.

Introduction

Anaerobic digestion will be required for the upgraded WWTP due to the need to treat primary solids. During anaerobic digestion, two distinct processes take place; acid fermentation and gas formation. These processes are accomplished by acid forming bacteria and methane forming bacteria respectively. The acid forming bacteria convert carbohydrates, proteins, and fats into organic acids and alcohols. The gas forming bacteria convert the organic acids and alcohols into methane and carbon dioxide. Maintaining the proper temperature, pH, mixing, and cell residence time will optimize the growth of these bacteria and the conversion of organic matter to the desired end products.

Two variations are used for the anaerobic digestion process; conventional and high-rate. In a conventional system, sludge is batch-loaded into the digestion vessel and allowed to stratify. No mixing or heat is provided. A high-rate system provides for mixing, heating, and a near constant supply of digestible material providing more favorable conditions for bacterial growth and digestion. The result is that the required detention time in the high-rate system is roughly one third that of a conventional system. A high-rate system is proposed for this project. High rate digesters can be significantly smaller than conventional designs due to the reduced detention time and increased solids loading rate. There are several variations related to digester design and operation that can be incorporated into a high-rate anaerobic system. These include the type of cover, the mixing system, the heating system, and the use of excess bio-gas.

The City currently uses the existing south primary clarifier and the secondary digester for storing digested sludge. As a part of the WWTP upgrade, the existing secondary clarifiers will be modified to provide additional sludge storage capacity. A field investigation will be undertaken early in the design process to determine if improvements to the South Primary Clarifier and the existing digesters will be required to facilitate continued use of these structures for sludge storage. Storage capacity is required during periods when wet weather prohibits land application.

Design Criteria

The WWTP upgrade will incorporate one new digester and piping stub-outs to accommodate a second digester for future conditions. The new digester will process a combination of primary sludge, thickened WAS and occasionally, septage from the City's STEP system. Table 12.1 shows the predicted startup and future sludge production values prior to digestion. The values shown in this table assume that 30% of the BOD and 60% of the TSS is removed in the primary clarifier and that the overall yield coefficient is 0.8 lb MLSS/lb BOD. The load contribution from STEP system septage is minimal accounting for less than 0.5% of the total BOD load to the WWTP. Therefore, septage loading is assumed to be included in the values shown on Table 12.1.

Table 12.1 – Sludge Production for the Tillamook WWTP

Item	Startup		Design	
	Average Month	Peak Month	Average Month	Peak Month
Raw Influent BOD (ppd)	1,250	1,485	2,100	2,475
Raw Influent TSS (ppd)	1,250	1,485	2,100	2,475
Primary Effluent BOD (ppd)	875	1,040	1,470	1,733
Primary Effluent TSS (ppd)	500	594	840	990
Primary Solids Concentration (%)	3.5	3.5	3.5	3.5
Primary Solids Volume (gpd)	2,570	3,050	4,230	4,990
Primary Solids VSS (%)	80	80	80	80
Primary Solids TSS (ppd)	750	891	1,260	1,485
Primary Solids VSS (ppd)	600	713	1008	1,188
WAS Solids TSS (ppd)	700	832	1,176	1,386
Thickened WAS Concentration (%)	5.5	5.5	5.5	5.5
Thickened WAS Volume (gpd)	1,530	1,810	2,560	3,020
WAS VSS (%)	80	80	80	80
WAS VSS (ppd)	560	670	940	1,110
*Combined Solids Load (ppd)	1,450	1,720	2,400	2,900
*Combined Sludge Concentration (%)	4.4	4.2	4.2	4.3
*Combined Sludge Volume (gpd)	4,100	4,860	6,790	8,010
*Combined Sludge VSS (%)	80	80	80	80
*Combined Sludge VSS (ppd)	1,200	1,400	1,900	2,300

* These values apply to digester influent. Digester effluent flowrates will remain unchanged from influent values, however effluent solids/sludge will be reduced due to VSS destruction within the digester. Anticipated VSS destruction and effluent sludge concentration are 50%-60% and 2.0%-2.6%, respectively.

Both the VSS loading rate and the EPA 503 Biosolids Rule must be considered to arrive at the required digester volume. The 503 Rule requires that Class B anaerobically digested sludge be held in the digester for a minimum of 15 days at temperatures between 95°F and 131°F to ensure adequate pathogen reduction. Compliance with the 503 Rule takes precedence over other design considerations since this is a regulatory requirement for land application of digested biosolids.

Metcalf & Eddy, Wastewater Engineering recommends VSS loading rates for high rate digesters of 0.10 lb/cf*day to 0.30 lb/cf*day. Based on past experience, Kennedy/Jenks Consultants has found that a VSS loading rate of 0.16 lb/cf*day optimizes digester performance and that loading rates lower than this may result in reduced overall VSS destruction, potential operation problems, and lower quality bio-gas production.

Meeting both the 15 day retention requirement and the recommended loading range of 0.16 lb/cf*day is not possible unless the combined primary solids and WAS can be thickened to an

average concentration of approximately 5% (assuming an average VSS concentration of 80% in the feed sludge). Since provisions will not be incorporated at Tillamook to thicken primary sludge, it will not be possible to achieve an average feed concentration of 5%. Therefore, the optimal feed value of 0.16 lb VSS/cf/day will not be met. However, thickening WAS to approximately 5.5% will bring the average feed concentration (WAS plus primary sludge) to approximately 4.3%, assuming the primary sludge concentration is 3.5%. This will result in the VSS Loading Rate range shown on Table 12.2. While not at the optimal 0.16 lb VSS/cf/day, these loading rates should allow for adequate digester performance. For increased future flexibility, piping provisions should be incorporated in the thickener installation to allow for thickening of primary sludge prior to digestion. Table 12.2 lists design criteria for the digester.

Table 12.2 – Digester Design Criteria

Criteria	Value
Design HRT (days)	15 (a)
Average VSS Loading Rate (lb/cf/day)	0.06 to 0.10 (b)
Peak Month VSS Loading Rate (lb/cf/day)	0.07 to 0.12
Effective Volume (gpd)	144,000
Diameter (ft)	34
Approximate Side Water Depth (ft)	20
Bottom Slope (in/ft)	3
Gas Production (cf/day) (c)	9,000 to 14,250 (d)
Minimum Temperature Requirement (°F)	95
Design Temperature (°F)	100
Normal Low Winter Air Temperature	40°F

(a) 15 days is the minimum retention time required to meet EPA Class B requirements for pathogen reduction.
(b) Based on Average Month "Startup" to "Design" loading rates, a 15 day HRT and, 80% VSS concentration.
(c) Digester gas normally contains about 65% methane.
(d) Based on Average Month "Startup" to "Design" loading rates. Assumes 15cf per lb VSS destroyed.

Digester Tank and Cover

The digester tank will be a conventional cast-in-place concrete design with a fixed steel or cast-in-place concrete cover. The tank should incorporate a sloped bottom to collect grit and other heavy inert material. The digester interior and cover will be coated to resist the corrosive environment.

Sludge feed will be done through the digester mixing piping. Sludge withdrawal will be through an overflow pipe located near the top of the digester. The tank will have an access staircase and a walkway on the cover to service gas handling appurtenances.

Reliability and Pumping Redundancy

The proposed digester mixing and heating system requires three pumps that are explained in greater detail below. Providing a duty pump and standby pump for each service is an option for added reliability. However, the piping, valving, controls, and additional building space required to provide an installed backup pump for each service will result in a significant additional cost and building space requirement. While Figures 12.1 and 12.2 show how a standby pump would be integrated within the system, it is not anticipated that they will be incorporated into the design. Having a spare set of pumps in storage will be a more cost effective solution to providing a reasonable level of digester reliability.

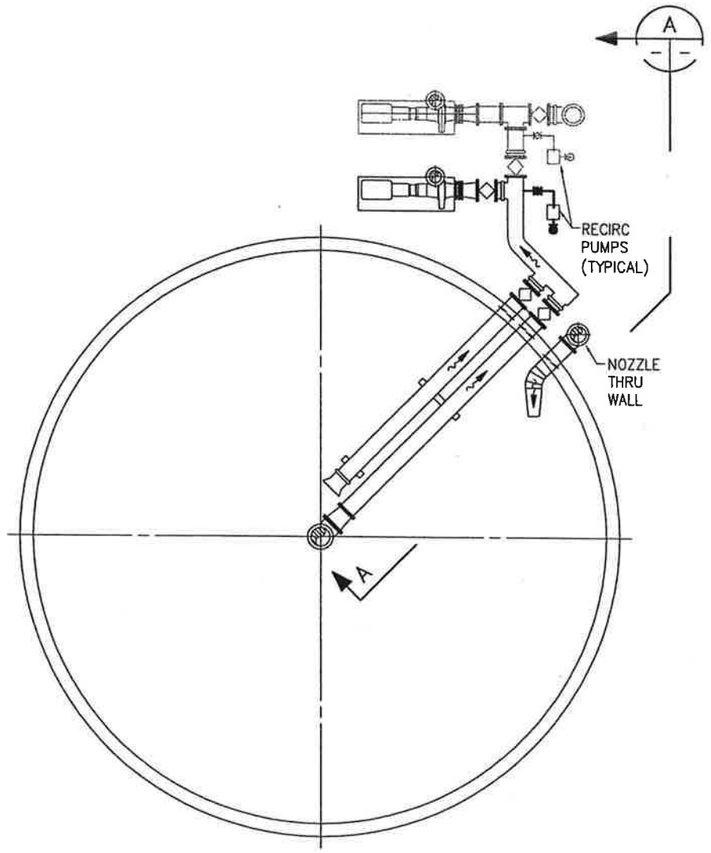
Digester Mixing System

The mixing system will use an externally located pump to circulate the digester contents. Externally pumped mixing systems are characterized by highly uniform mixing and offer a number of advantages over in-vessel mechanical mixers and gas recirculation mixing systems. The system can be alternately set to draw off either the top or bottom of the tank. The pump circulates the liquid back into the tank through a high velocity nozzle. Mixing energy developed at the discharge nozzle creates a spiraling vortex within the digester resulting in better overall mixing throughout the tank. Better mixing results in higher volatile solids destruction, increased gas production, and more complete digestion of fats, oils, and grease. Since this system can be used to draw off the bottom of the digester, smaller grit particles can be maintained in suspension. By regularly drawing off the top of the tank, the scum blanket that normally forms in an anaerobic digester can be incorporated into the liquid portion and used as high energy food for the biomass. The alternating mixing strategy prolongs the period between cleaning and increases the available volume in the digester. More efficient mixing also provides for better temperature distribution within the digester resulting in better overall digestion. A simplified plan view of the mixing system is shown on Figure 12.1.

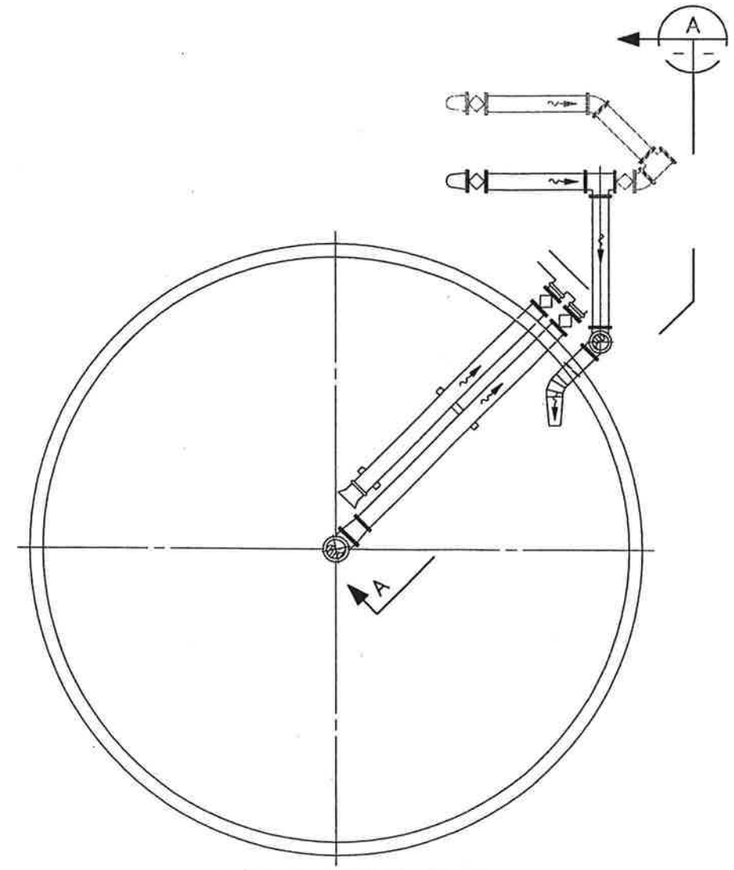
Mixing System Components

The mixing system is shown schematically on Figure 12.2. Piping and valves will be provided to allow the pump to draw from either the bottom of the tank or near the liquid surface. Discharge from the mixing pump into the digester is through one of two nozzles located near the top and bottom of the digester wall. These upper and lower suction and discharge locations are provided so that the mixing patterns in the digester can be alternated periodically by changing valve positions. Primary sludge and WAS are fed to the digesters through the mixing system.

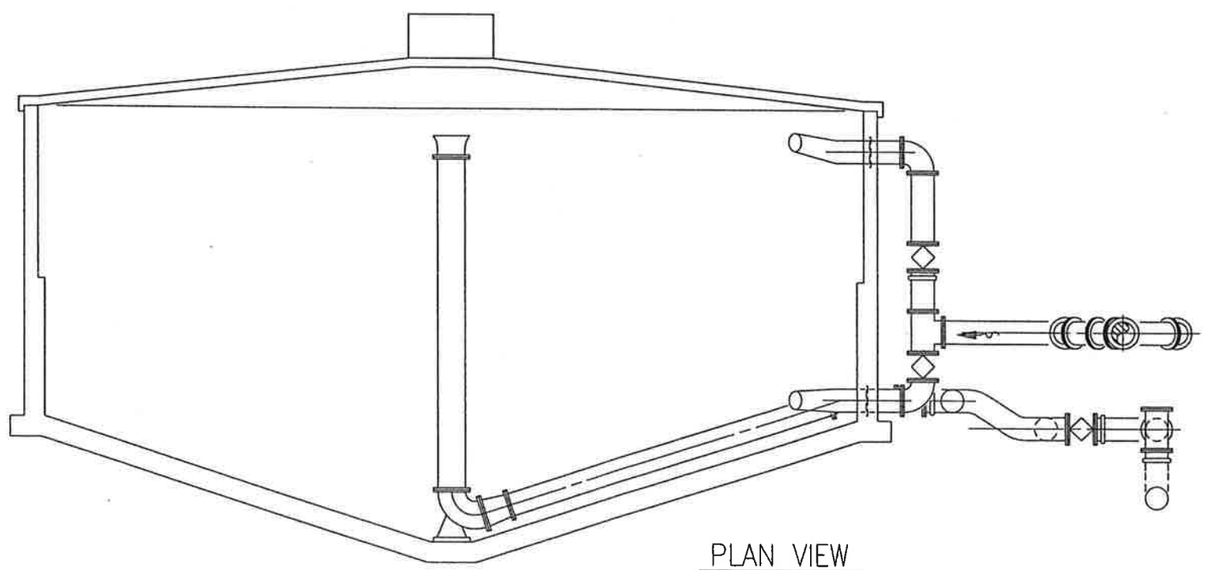
The mixing system will be sized to provide about 10 volumetric turnovers of the digester per day. This equates to a pump rate of 660 gpm for the digester proposed for this project. The velocity at the discharge nozzles should be approximately 12 fps. The diameter of the mixing pipes should allow for velocities between 6 to 8 fps. In general, about 6 hp of pumping capacity is required per 100,000 gallons of digester volume.



LOWER PIPING PLAN
SCALE: NTS



UPPER PIPING PLAN
SCALE: NTS



PLAN VIEW
SCALE: NTS

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City of Tillamook, Oregon

Anaerobic Digester Conceptual Layout Plans and Section

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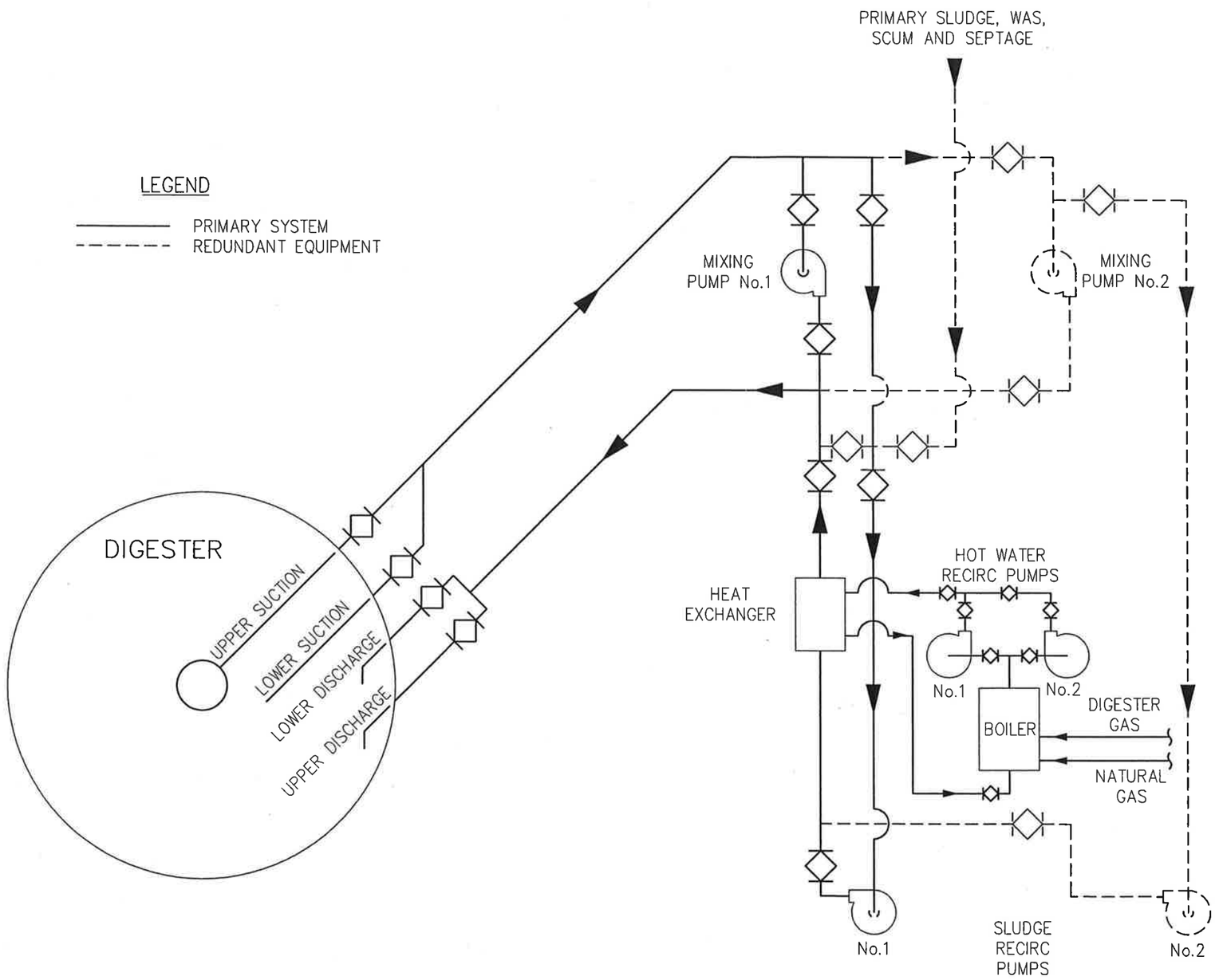
Figure 12-1



Kennedy/Jenks Consultants

City of Tillamook, Oregon

Anaerobic Digester
Mixing and Heating Schematic

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Figure 12.2

Axial-flow, propeller, torque-flow centrifugal, screw centrifugal, and chopper pumps have all been used successfully for mixing and recirculation service. Horizontal screw centrifugal or chopper pumps such as the Wemco Hidrostal or Vaughn Chopper are preferred for applications where rags may be present in the digester contents. One of these pump types will be used as the basis of design for the mixing pump and the sludge recirculation pump.

Digester Heating System

The heating system is shown schematically on Figure 12.2 and consists of a spiral heat exchanger, a sludge recirculation pump, a hot water recirculation system, and a boiler. The boiler will primarily be fueled with digester gas and have a backup supply of natural gas for times when digester gas production is insufficient to maintain the required digester temperature. At a minimum, the new heat exchanger would be sized to transfer sufficient heat to maintain the digester at 100ºF during the coldest winter month. The minimum required temperature for meeting Part 503 regulations is 95°F.

The boiler recirculation system circulates heated water between the boiler and the heat exchanger. The boiler should be capable of heating the recirculation water up to 190°F. An automated control system will be used to maintain the temperature setting of the feed water to the heat exchanger. A sludge recirculation pump will be used to circulate sludge through the heat exchangers. Recirculating sludge is drawn from the header on the suction side of the mixing pump, pumped through the heat exchanger and then discharged back into the mixing system through a header on the discharge side of the mixing pump. The sludge recirculation pump must be capable of overcoming the headloss incurred in the heat exchanger and the line pressure produced by the digester mixing pumps.

The spiral heat exchanger uses two concentric spiral channels to transfer energy from the heated water to the sludge. The spiral design provides optimum heat transfer, lower headloss, and minimizes the overall size of the unit when compared to flat plate exchangers. This type of heat exchanger has proven to be efficient and reliable for this application. The basis for design will be spiral heat exchanger from Alfa Laval Thermal.

A 500 gallon storage tank for natural gas will be installed. This should provide for approximately one week of supplemental heating capacity. The Parker Boiler model WH490 or WH730 will be used as the basis of design.

Digester Equipment Location

A new Solids Processing Building will be constructed to house the digester mixing and heating systems. The building is discussed in Technical Memorandum 13.

Sludge Storage

Digested sludge storage will be provided by utilizing the existing south primary clarifier, the existing primary and secondary digesters, and the existing secondary clarifiers for this purpose. The WWTP currently uses the existing south primary clarifier and the secondary digester for sludge storage. The existing sludge piping will need to be field evaluated and documented to ensure that it is sufficient to meet the requirements of the upgraded treatment plant. It is anticipated that digested sludge can be transferred by gravity from the new digester to any of the five storage basins. With the exception of the two existing secondary clarifiers, transfer of sludge between the storage basins will be done using the existing sludge piping and the progressing cavity pumps in the existing primary clarifier pump room. Since the water surface of the existing secondary clarifiers is below grade, it will be necessary to pump the sludge out of these basins. A schematic of the sludge pumping and storage system is shown on Figure 12.3.

Early in the design process, a field investigation and consultation with City staff should be undertake to determine if any modifications are required to facilitate the continued use of the south primary clarifier and the existing primary and secondary digesters. Improvements or repair of the transfer piping may be required.

Decanters will be installed in the South Primary Clarifier to concentrate stored sludge and to remove rainwater. Mixing units will be installed to mix the basin contents prior to sludge transfer. Submersible, floating, and platform-mounted mixers will be evaluated.

The existing secondary clarifiers will be converted for use as sludge storage basins. Each basin will provide approximately 117,000 gallons of digested sludge storage capacity. The internal mechanism and walkway will be removed. Mixing units will be installed to mix the sludge prior to transfer. The existing underflow pump will be evaluated for suitability to pump stored sludge to the sludge loading dock or to the other storage basins. There is an identical spare pump, stored onsite that was furnished as part of the 2003 CAP upgrades.

Calculations indicate that approximately 4,100 gpd of digested sludge will be produced at startup increasing to 8,000 gpd at the limits of design. These values assume that supernatant is not removed during storage. The capacities of the structures designated for sludge storage are shown on Table 12.3. Assuming that decanting is not implemented, these structures would provide approximately 115 days of storage at startup and approximately 59 days at the design value. If decanting of supernatant is implemented, the storage capacity could theoretically be extended by up to 30%. Provisions for decanting will be incorporated into the new sludge storage tanks.

Table 12.3 – Digested Sludge Storage Volume

Structure	Volume (gallons)
Existing Secondary Clarifiers	235,000
Existing Primary Digester	67,000
Existing Secondary Digester	84,000
Existing South Primary Clarifier	85,000
Total Storage Capacity	**471,000**

Technical Memorandum No. 13

Subject: Solids Processing Building/Blower and Standby Generator Building
Building Description and Preliminary Code Review
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted by: Dan Wright, AIA. and Harry Ritter, P.E.

PART 1 - SUMMARY

1.01 Purpose

This review is divided into two sections for code review purposes. Occupied structures and unoccupied structures are governed by different regulations.

- Occupied Structures as defined by the building codes are governed by the 2004 Oregon Structural Specialty Code, International Fire Code and National Electrical Code.
- The National Fire Protection Association, National Electrical Code and OSHA generally regulate site structures.

1.02 Codes Used for this Evaluation

- 2004 Oregon Structural Specialty Code (IBC) with amendments to the 2003 International Building Codes (IBC), Mechanical Code (IMC), Plumbing Code (IPC), Fire Code (IFC), 2003 editions
- 2005 National Electrical Code (NEC)
- National Fire Protection Association Codes (NFPA), particularly NEC 820
- Federal OSHA Standards

1.03 Permitting Agency

Permitting is handled by Tillamook County

- Phone: 503-842-3408
- Website: www.co.tillamook.or.us/gov/ComDev/Building

1.04 Site

The expansion site, referred to as the North Site, is bordered by the Trask River on the west side, by 3rd Street on the North side by residential and multi-occupancy dwellings on the east site, and by the existing WWTP on the south side. The existing treatment plant is located within City limits at 710 5th Street, Tillamook, OR, 97141. The North Site is approximately 2.3 acres. One of the buildings discussed in this memorandum (Blower Building) will be constructed on the North Site. The other building (Solids Processing Building) will be constructed on the existing site.

PART 2 - OCCUPIED STRUCTURES

2.01 General Descriptions

Two new buildings and several basins are planned as part of the WWTP expansion. The exterior of the buildings will generally match the existing Administration/Laboratory building in appearance. The overall style of the buildings will be utilitarian, designed to meet the functional requirements at minimum cost.

In addition to several new treatment basins, these buildings are proposed for the project as follows:

- Solids Processing Building
- Blower Building

2.02 The Solids Processing Building

The building will consist of five rooms housing the solids thickening equipment (Solids Thickening Room), the anaerobic digester appurtenances (Digester Appurtenances Room), the boiler (Boiler Room), the dewatering polymer (Polymer Room), and a motor control center (MCC Room). A floor plan of the building is shown on Solids Processing Building, Figure 13.1. The building has the following features:

- Estimated Floor Area: 1,190 gross square feet
- Height: Single story, not exceeding 35 feet at the highest point.
- Construction: IBC Type V-B, slab-on-grade design with split-faced concrete block walls. The roof surface will be a Class “A” standing seam metal roof assembly.
- Occupancy Type: F-1
- Access: The digester appurtenance and MCC room will be accessed through single or double swing personnel doors as appropriate. Swinging and overhead doors will be used to provide access and ventilation to the thickening equipment room
- Material Storage: Dewatering polymer; non-hazardous.
- Hazardous Materials utilized: Propane and digester gas (80 percent methane) piped to boiler. Methane quantities used will be below exempt quantities in IFC Table 2703.1.1. Propane tank shall be separated from occupied buildings and property line by a minimum of 25 feet. Refer to discussion in paragraph 2.08.
- Drainage: Floors in the building will be sloped and drained to the recycle pump station which will pump back to the treatment process. The recycle pump station, to be constructed outside the building, will pump thickener filtrate to the treatment plant.
- Ventilation and Heating: Heating in Thickening Room only. All other spaces ventilated. No cooling.
- Electrical: The Boiler Room is required to meet Class 1, Group D hazard classification.
- Occupant Load: 3

- Occupancy Separation: The building contains a Boiler Room that requires a one-hour fire separation per IBC Table 302.1.1

2.02, A. - Description of Rooms

Sludge Thickener Room: The Sludge Thickening Room will house the mechanical thickener and control panel. The room housing the thickening equipment does not require an NEC electrical hazard classification per NFPA 820. Space will be heated.

Digester Appurtenances Room: The Digester Appurtenances Room will contain one digester mixing pump, one digester recirculation pump, and a spiral heat exchanger. The room housing the thickening equipment does not require an NEC electrical hazard classification per NFPA 820. Space will be heated.

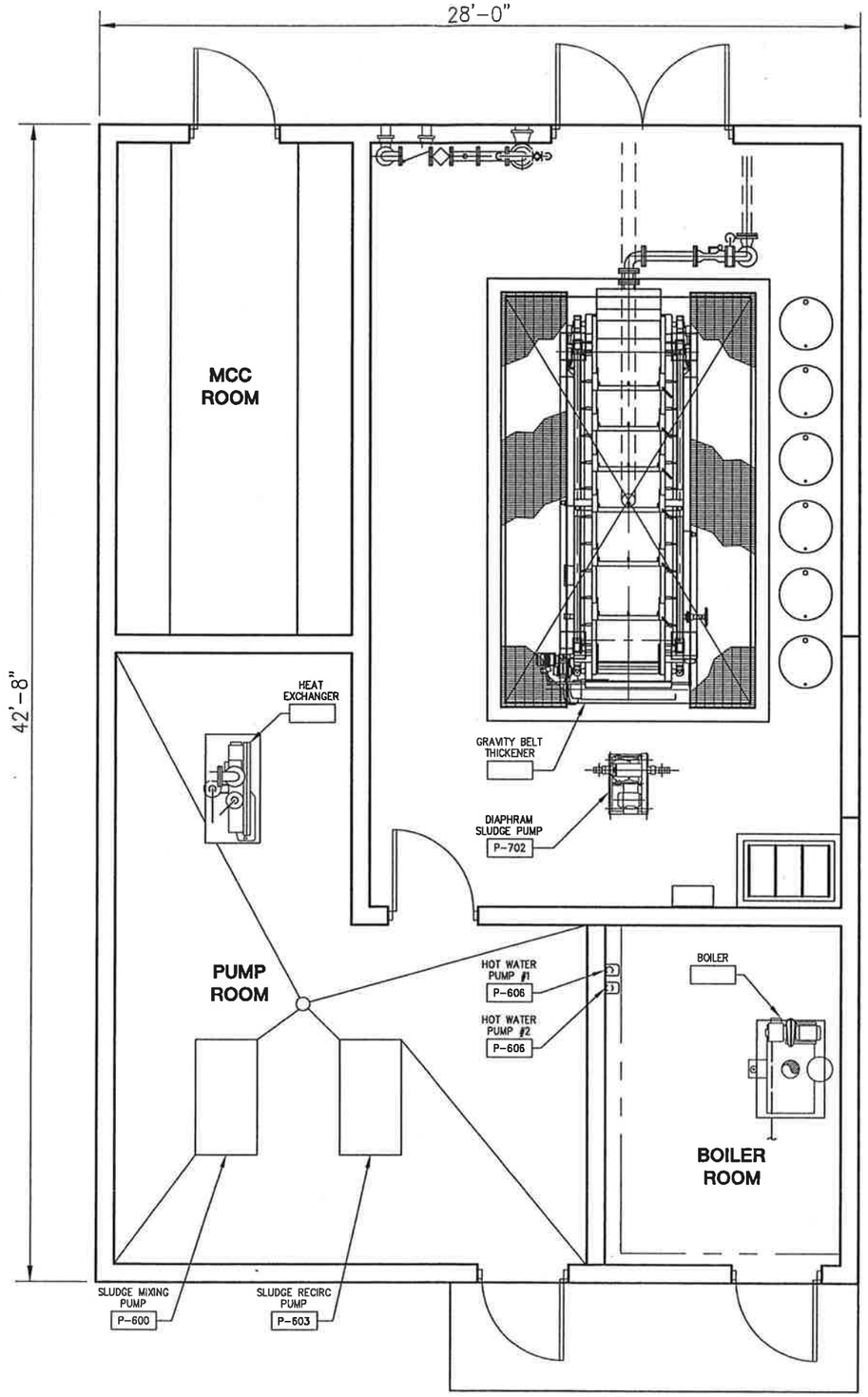
Boiler Room: The boiler will primarily use waste digester gas for heating. Propane, stored in an exterior tank, will be used as a supplemental energy source for boiler heating. This room, since it contains the gas handling equipment and boiler, shall be classified as a Class I, Group D Hazardous Location and will need a combustible gas detector. This room cannot have any unprotected openings into adjoining interior spaces and the exterior doors into this room must be located 10-feet or more from the wall of any adjoining spaces.

Motor Control Center Room: Motor control for the thickening equipment and the digester appurtenances will be centralized in this room. Lighting and instrumentation panels will also be located in this room. Total power is less than 1,200 amps.

2.03 Blower and Standby Generator Building

The building will consist of three rooms; the Blower Room, the Generator Room, and the MCC Room. The floor plan of the building is shown on the Blower and Standby Generator Building Plan, Figure 13.2. The building has the following features:

- Estimated Floor Area: 1,100 gross square feet
- Height: Single story, not exceeding 35 feet at the highest point.
- Construction: IBC Type V-B, slab-on-grade design with split-faced concrete block walls. The roof surface will be a Class “A” standing seam metal roof assembly.
- Occupancy Type: F-1
- Access: The blower, generator, and MCC rooms will be accessed through single or double swing personnel doors as appropriate.
- Material Storage: Diesel fuel will be stored in a double-walled tank exterior to the building. No other chemicals will be stored.
- Drainage: Floors in the building will be sloped and drained to the sanitary sewer which will drain to the influent pump station. Drainage will not ordinarily be generated in this building.



PLAN VIEW
SCALE: 3/16"=1'-0"

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City of Tillamook, Oregon

Solids Processing Building Floor Plan

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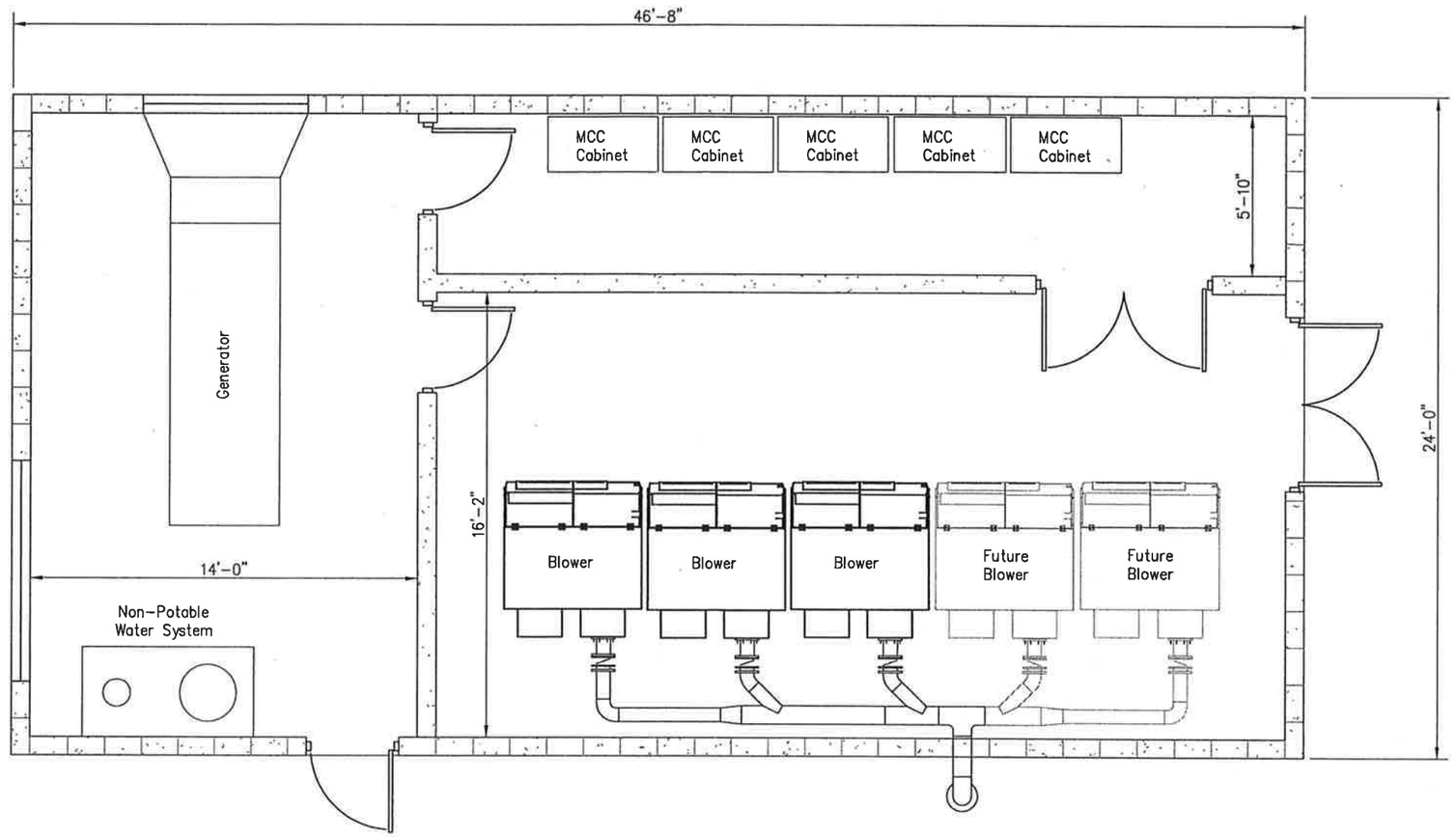
Figure 13.1

P:\ccd\04\0491010.20_WV IGN\Final Predesign Fig 2006\Fig 13-1 Solids Bldg Floor Plan.dwg Feb 01, 2006 - 3:05pm



PLAN VIEW
SCALE: 3/16"=1'-0"

Kennedy/Jenks Consultants

City of Tillamook, Oregon

Blower Building Floor Plan

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Figure 13.2

- Ventilation and Heating: Heating in Thickening Room only. Pending heat calculations, cooling may be required in the MCC room.
- Electrical: All rooms are rated as non-explosive environments.

2.03, A. - Description of Rooms

Blower Room: The Blower Room will have aeration blowers installed along one wall and be separated from the other rooms by a CMU wall. Air will be drawn into the room through louvers located in the wall adjacent to the Blowers. Controls for the blowers will be housed on the opposite wall. The room will not require heating, or cooling.

Standby Generator Room: The Generator Room will house a 400kW standby generator. An intake and exhaust louver will provide for ventilation of the generator. Heating and cooling will not be required. Exhaust from the generator will be discharged to outside the building. Fuel for the generator will be stored outside the building in a dual-wall tank.

MCC Room: The MCC Room will have cabinets installed along one wall. This room may require cooling. The MCC room will house panels to service the blowers, the influent pump station, the secondary clarifiers, and the RAS, WAS, and scum pumps as well as lighting panels and other miscellaneous equipment. The automatic transfer switch for the standby generator will be housed in this room.

2.04 Handicap Access

Open to interpretation. Main entrance required to be accessible in F occupancies (IBC 1106). Equipment spaces need not be accessible (IBC 1108.4.14).

2.05 Construction Materials

The buildings can be built of any materials approved by the International Building Code (wood, concrete, masonry or steel).

2.06 Fire Sprinklers

None required by occupancy or building area.

2.07 Storage

Storage consists of hazardous chemicals and small quantities of non-hazardous maintenance materials.

2.08 Physical and Health Hazards

2.08, A. - Interior Storage (these chemicals are in the Solids Processing Building)

Methane: 500 to 850 cubic feet produced per hour. No gas is stored. The gas is fed to a boiler for immediate use. CAS No. 74-82-8, CAA Section 112 threshold is 10,000 gallons. (NFPA/IFC Rating: H4, Flammable Gas) Classified per IFC as meeting Flammable Gas category. The chemical used in the system is below the exempt (1,000 cubic feet) amount per IFC Table 2703.1.1 (1)

Polymer: 165 gallons maximum stored in 55-gallon drums; Specific chemical properties (MSDS sheet) should be reviewed for safety issues. Polymers in emulsion are generally flammable while dry and solution polymers are not flammable. Polymers when wet are extremely slippery. Regardless of the specific polymer type the quantity stored is below the exempt amount per IFC Table 2703.1.1 (1).

2.08, B. - Exterior Storage

Propane (liquefied gas): 500 gallons (125,000 cubic feet at 1 atmosphere) stored in a single tank. CAS No. 74-98-6, CAA Section 112 threshold is 10,000 gallons. (NFPA/IFC Rating: H4, Flammable Gas) Classified per IFC as meeting Flammable Gas category. The chemical stored exceeds the exempt amount (30 pounds) per IFC Table 2703.1.1 (2). The gas storage will comply with IFC Chapters 27, 30 and 35.

2.08, C. - Chemical Separation

The propane and diesel fuel storage areas are remote from the other buildings, property line and chemical storage areas. Fuel tanks shall be located 25 feet away from property line and buildings (IFC Table 3504.2.1).

2.08, D. - Spill Control and Containment

Not applicable for exterior propane tank. The diesel tank will be stored in tank providing dual-containment.

2.08, E. - Liquid level Limit Controls (IFC 2703.2.7)

Tanks containing hazardous liquids in excess of 500 gallons shall be equipped with a liquid level limit control to prevent tank overfilling.

2.09 Building Construction

2.09, A. - Exterior Walls

The exterior walls are not required to be fire resistive construction since the buildings are more than ten (10) feet from property lines. Openings do not need to be protected if less than 15 percent of wall area. See Note 2 below the Code Evaluation Tables.

Based on the current configuration it is recommended that the buildings be a minimum of 10 feet from the assumed property line to avoid additional construction costs associated with fire resistive construction.

2.09, B. - Interior Walls

A one-hour fire-rated occupancy separation wall assembly is required between the Boiler Room and other portions of the building. Otherwise no special construction or fire resistance required.

2.09, C. - Energy Efficiency Regulations

Required in all conditioned spaces (heated or cooled) and in accordance with IBC Section 1312. Exterior walls do not need to be insulated if heating system is designed to keep the space at 45 degrees or lower (IBC 1312, exception 1)

2.10 Safety Features

- Recommend that areas where polymers are dispensed should be finished in a slip resistant grit type coating or grating.
- Emergency eyewash and showers are required at storage areas containing liquids that constitute a health hazard per OSHA.
- Fire extinguishers required in each room.

2.11 Systems, Equipment, and Processes

Chemical storage tanks shall be designed and constructed in accordance with nationally recognized, approved standards (API 650, UL 142, ASME Standards, ANSI B31, B31.3) (IFC 2703.2.1).

Chemical piping shall be designed and constructed in accordance with IFC paragraph 2703.2.2

2.12 Identification Signs

- Tanks and piping shall bear hazard identification signs per NFPA 704 (IFC 2703.5).
- Piping shall bear hazard identification signs per ANSI A 13.1 (IFC 3003.2.3).
- Additional signs are required at entrances to locations where hazardous materials are stored.
- No smoking signs shall be provided on all doors and accessible openings into areas with flammable materials.
- No smoking signs shall be provided within 25 feet of outdoor storage areas.

2.13 Security

Hazardous material storage areas shall be secured against unauthorized entry (IFC 2703.9.2).

2.14 Vehicular Protection

Guard posts shall be provided to protect storage tanks and connected piping subject to vehicular damage (IFC 2703.9.3). Posts shall be:

- 4 inch diameter minimum and concrete filled.
- Spaced no more than 4'-0" O.C.
- 3 feet deep minimum in concrete footing of no less than 15" diameter.
- Top of post no less than 3 feet aboveground.
- Located no less than 5 feet from the tank.

2.15 Electrical

- Installed wring and equipment in accordance with the National Electrical Code and adopted State of Oregon amendments. (IFC 2703.9.4)

- Lighting protection is recommended per NFPA 820, Section 5-7.2 and installed in accordance with NFPA 780.

2.16 Setbacks

The building setback is governed by the planning (zoning) regulations. It is not anticipated, given the location on site and the size of the buildings, that planning issues will affect the design.

2.17 Noise

Community Noise Regulations were not reviewed.

PART 3 - CODE REFERENCE

Table 3.5 – IBC References

Item	IBC Reference unless noted otherwise	Comments
Construction Type	IBC 602.5, Type V-B	Any materials allowed by Code.

Item	IBC Reference unless noted otherwise	Comments
Basic Allowable Area	Table 503	Basic allowable Area = 8,500 SF
IBC Construction Requirements		
Height	Table 503	one story
Walls location from property line	Table 602	
Openings	Table 704.8, footnote g	No protection required
Occupancy Separations	Table 302.3.2	Not Applicable, One occupancy
Egress/Exiting		
Occupant Load	IBC Table 1004.1.2	
Electrical Room	NEC	Two exits required if power exceeds 1200 amps
Exits required	Table 1014.1	The building has 1 exit per room, minimum.
Sprinklers	IBC 903.2.3, Group F-1	
Spill Control	IBC 414.5.5,	Not applicable to gases
Secondary Containment	IBC 414.5.5, IFC Table 2704.2.2	Not applicable to gases
Fire Water Containment	IBC 414.5.5	Not Applicable
Handicap Access	IBC 1106	
Fire Alarm	IFC 907.2	Not required in one story F with fewer than 500 occupants
Emergency Alarm	IFC 908, IFC 2704.9 & 2705.4.4	Not required, below exempt amounts & transferred in piping
Smoke Detectors	IFC 907.2	Not required in one story F with fewer than 500 occupants
Smoke Control	IFC 910.2 & Table 910.3	Not required, Area less than 50,000 SF
Fire Extinguishers	IFC 906.1	Required

Item	IBC Reference unless noted otherwise	Comments
Exit Signs	IBC 1011.1	Not required per IBC 1011.1, exceptions 1 and 2
Standby or Emergency Power	IFC 414.5.4 & IFC 2704.7 & 2705.1.5	Not applicable
Ventilation	IFC 2704.3 & 2705.1.9	Not Applicable since quantities are below exempt amounts, however room shall be ventilated per IBC and IMC for standard use.
Attic Access & Ventilation	IBC 1209.2	Required when attic is enclosed & wood framed & is 30” high or higher. Min. size of opening 22”x30” required into attic space
Access to Building Equipment in Attic	IBC 1209.2	Ladder Access OK. Note equipment on roof must be accessible by fixed ladder or stair per IMC

1. The buildings may be placed on the property line if it is constructed to meet fire resistive requirements. Refer to IBC Table 602 if the building is less than 10 feet to the property line. The requirements become more restrictive (and expensive) as the distance to the property line decreases.

2. Refer to the International Building Code and International Fire Code for the exact definition of property line. The property line location for code purposes is not always the legal property line and may vary based on the surrounding uses.

PART 4 - PROCESS STRUCTURES

4.01 Description

These structures are not occupied and most are not covered. The following table indicates the NFPA and NEC requirements for these functions.

- Influent and Recycle Pump Station
- Fine screen and grit removal
- Aeration Basin
- Flow control structure
- Secondary Clarifiers
- RAS/WAS/SCUM Pump Station and Valve Vault
- Anaerobic Digesters

- Chlorine Contact Tanks
- Sludge Storage

Table 13.6 - Process Structure Code Requirements

ID No.	Location and Function	Ventilation	Classification Area	NEC Classification	Construction Material	Fire Protection
1	Wastewater Pump Station Wet Wells (includes both Influent PS and recycle PS)	Not ventilated	Entire space	Class I, Division 1, Group D	NC, LC, or LFS	CGD
2	Headworks Screen and Grit Removal Systems	Open to atmosphere	Within 10 feet envelope around equipment and open channel.	Class I, Division 2, Group D	NC, LC, or LFS	FE, H
3	Aeration Basin	Open to atmosphere		Unclassified		H
4	Flow Control Structures	Open to atmosphere		Unclassified		H
5	Secondary Clarifier Tanks	Open to atmosphere		Unclassified		H
6	Scum pits, RAS and WAS Pumps at Clarifiers	Not enclosed, open to atmosphere	Within 10 feet envelope around equipment and open channel.	Class I, Division 2, Group D	NC, LC, or LFS	H & FE
7	Anaerobic Digesters	Not ventilated	Enclosed space	Class I, Division 1, Group D	NC	H & FE
8	Chlorine Contact Tank	Not enclosed, open to atmosphere	Within 10 feet envelope around equipment and open channel.	Unclassified	NC	H & FE

ID No.	Location and Function	Ventilation	Classification Area	NEC Classification	Construction Material	Fire Protection
9	Covered Sludge Storage Basins	Ventilated	Enclosed space	Class I, Division 1, Group D.	NC	H & FE
10.	Uncovered Sludge Storage	Open to atmosphere		Unclassified		H

LEGEND FOR SITE STRUCTURES (Fire Protection Column)
CGD: Combustible gas detection system
FE: Portable Fire Extinguisher
H: Fire Hydrant

Technical Memorandum No. 14

Subject: Septage Receiving Station
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Tim Caire, P.E.

Introduction

A septage receiving station will be incorporated into the expansion of the Water Reclamation Facility to remove plastics, grit, and other inert material prior to pumping the septage to the digester. The septage receiving station will also be used to process material collected in the City's Vactor truck during routine line flushing.

- This Technical Memorandum provides a brief discussion of the sources, volumes, and characteristics of septage which are generated within the Tillamook service area. It also provides a description of proposed new septage facilities, including utilization of existing plant facilities, and associated site layout considerations.

Background

Currently, there are 63 septic tank effluent pumping (STEP) systems located along North Highway 101 which discharge to the City's sanitary collection system. The City of Tillamook 2005 Wastewater Facility Plan contains detailed data on existing STEP systems. Periodically, the material that accumulates in the bottom of the tank is pumped out and trucked to the treatment plant for disposal. In addition to the solids accumulated in the bottom of the tank, the contents of the tank will include the liquid portion of the tank at the time of pumping. The combination of this material is referred to as '"septage". Grease from food preparation is often present in the septic tanks.

The septic tanks must be evacuated at a frequency which is dependent on the degree of use, and also on the type of waste collected. Some of the heavily used tanks require pumping every two to three months while others may only require pumping once per year. On average, it can be assumed that all of the tanks are pumped twice per year. Most of the tanks have a capacity of 1,500 gallons. Routinely, the operator will use the STEP pump to pump a tank down to about 700 gallons and then transfer the remaining liquid into the City's 1,500-gallon Vactor truck. The operator will normally try to pump two tanks per trip.

In the past, the City has dumped septage collected from the STEP tanks directly into the influent pump station at the treatment plant. This has, on occasion, upset the biological treatment process and has also resulted in the accumulation of plastic material in the digested biosolids. Plastic that accumulates in the biosolids must be removed before this material can be land applied resulting in an ongoing and extensive maintenance operation for the plant. The proposed receiving facility will reduce the labor involved with processing septage by removing plastics and other inert material before treatment and will allow the septage to be directly pumped to the anaerobic digester to eliminate the risk of a biological upset in the treatment plant. The receiving station will also allow for the segregation of plastics and inert material from

waste water accumulated during routine pipe flushing. Since the flush water is dilute with regards to BOD and other constituents, it will be introduced into the main treatment facility (at the lower reach of the headworks channel and downstream of the influent sampler), rather than into the digesters. An existing submersible pump station (the Supernatant Pump Station) will be incorporated into the design of the septage receiving station, as discussed below.

Septage Characteristics

Based on recent sampling data, the concentration of BOD, TSS, and ammonia in the tanks ranges from normal values found in domestic wastewater (i.e. 250 mg/l, 200 mg/l, and 35 mg/l respectively) to concentrations up to five times larger. This variability is due to the large diversity of users, which range from domestic users to commercial retailers and restaurants.

Septage is usually anoxic, therefore, it contains a very low concentration of dissolved oxygen, and often it also contains hydrogen sulfide. As a result, the microbial population which is acclimated to this environment and substrate is not well-suited for the aerobic processes of the main treatment facility. These factors, in addition to others, suggest that septage is best treated by the City's anaerobic digester.

Septage Facility Design Considerations

- The facility will contain adequate storage capacity, ease of pumper truck unloading, comminution of solids, and transfer of septage to the digesters or the plant headworks.

- The system capacity requirement is defined by the tanker truck capacity of 1,500 gallons. Freeboard is also required, thus a tank size of 2,000 gallons is proposed. (Note that the goal of a semiannual STEP system pumpout (on average) translates to two or three trips per week.)

- The receiving station will be capable of removing hard particulate materials including small rocks and plastic items. It will also provide a spray wash to facilitate passage of soft organics through a screen in order to segregate this material from hard particulates. Soft organics will be routed to the septage holding tank, along with dissolved material in the liquid septage and washdown water. Note that the capacity of the receiving station equipment is relatively large, e.g. 400 gpm for one candidate manufacturer; thus it will handle 1,500 gallons in approximately five minutes.

- The receiving station will provide washdown equipment and watertight truck hose connections with quick-disconnect couplings.

- Delivery of septage from the holding tank to the digester will be carried out at a low flowrate in order to minimize the impact on the digestion process. It is anticipated that

the flowrate will be between 10 gpm and 30 gpm. If suitable, the existing pump in the existing Supernatant Pump Station will be utilized for septage transfer.

- The septage receiving station will contain a metering and control system for monitoring septage flow.
- A high water level alarm will be provided for both the holding tank and the receiving equipment.
- Chemical feed facilities will not be provided at this time.

Description of Septage Facilities

The receiving equipment is a package unit. It is manufacturer by JWC Environmental, Lakeside, and other companies. Septage will flow by gravity from the Vactor truck to the septage receiving equipment via a 6-inch hose. The receiving equipment comminutes the process flow and then separates hard solids from soft solids and liquid, as discussed above. JWC Environmental offers an optional rock trap upstream of the comminutor. Hard solids are collected in a 55-gallon drum. The liquid stream flows by gravity to the septage holding tank. It is then routed to the digesters by a transfer pump. The pump discharge piping will be bifurcated and valved to allow for routing pipeline flush water to the plant headworks.

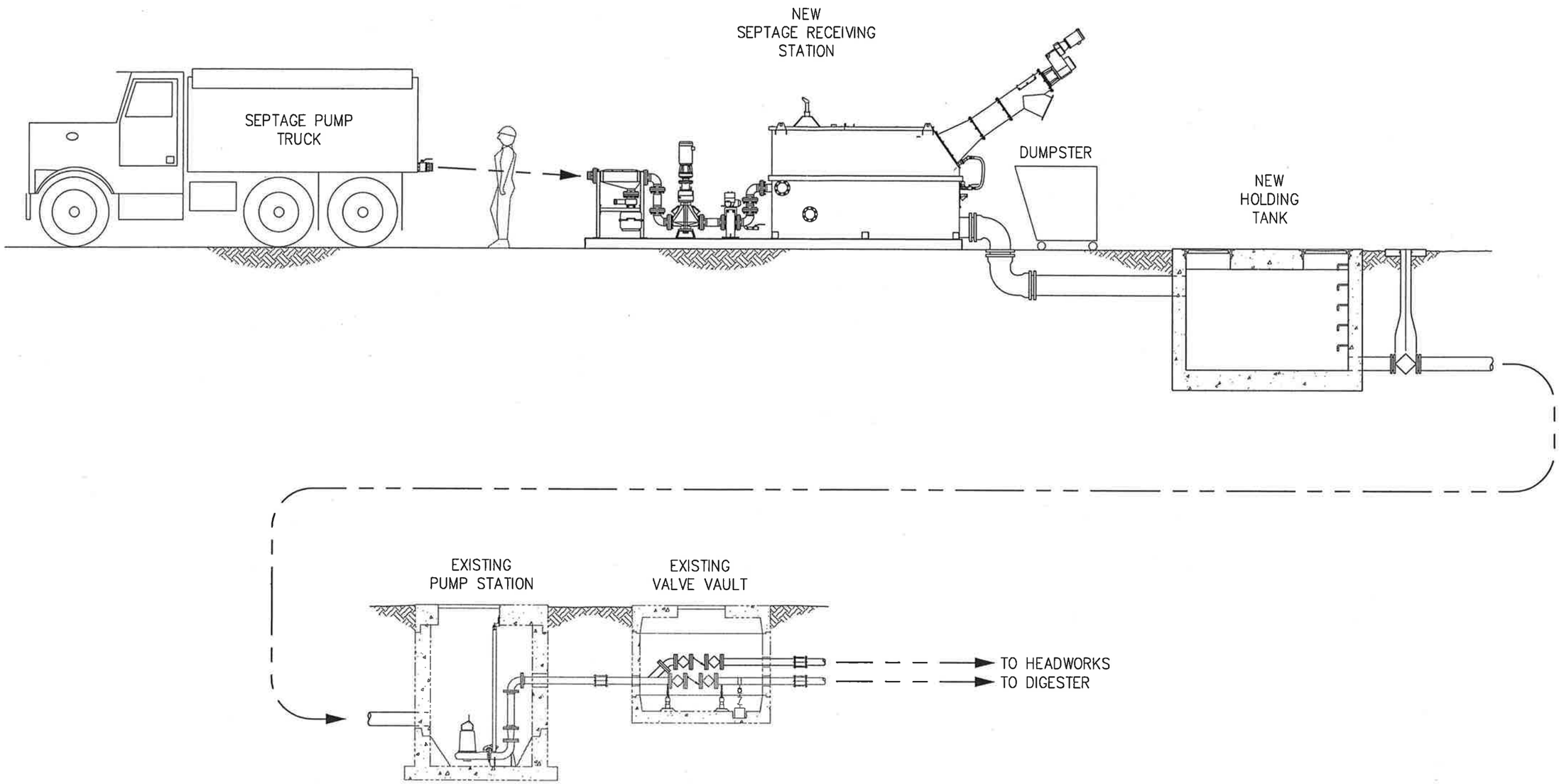
The septage receiving equipment will rest on a pad of approximate size 10 ft by 20 ft. Any spillage and washdown water is captured on the sloped pad and routed to a trench drain which empties into the holding tank.

The 2,000-gallon holding tank will be constructed either of large-diameter sewer pipe or a precast utility vault. As discussed, the existing Supernatant Pump Station could serve as the septage transfer pump station (it will be necessary to verify the performance of the existing pump for this application). The depth of the existing Supernatant Pump Station will limit the depth of the new holding tank to approximately 6.5 ft because if the water level in the holding tank were to fall below the transfer pump elevation, it would not be removed. Thus the approximate exterior dimension of the holding tank is 6 ft by 11 ft (if a utility vault is utilized). All tanks and vaults will be H-20 traffic rated, and will feature H-20 rated hatches.

The proposed system is shown schematically on Figure 14.1

Site Layout Considerations

The location of the existing Supernatant Pump Station is well suited for building the Septage Receiving Station. The site has ample room for installation of the storage tank and receiving equipment and provides easy access for the City's Vactor truck.



Kennedy/Jenks Consultants

City of Tillamook, Oregon

Septage Receiving Station Schematic

0491010.20

Figure 14.1

Technical Memorandum No. 15

Subject: Electrical, Instrumentation and SCADA Systems
City of Tillamook WWTP Expansion
K/J 0491010.00

Submitted By: Paul Reardon, P.E., Harry Ritter, P.E., and Tim Caire, P.E.

Purpose

The purpose of this technical memorandum is to describe the electrical, instrumentation and SCADA system improvements that will be incorporated into the upcoming WWTP expansion. Background information and recommendations for additional investigation are provided.

Background

Kennedy/Jenks Consultants recently completed a Wastewater Facility Plan for the City's treatment plant, and conducted a walkthrough of the Plant on 16 May 2005 with specific emphasis on the requirements of the electrical systems. At present, the Plant's power feed connection from the TPUD is rated at 230/120 VAC, 3-phase. There is a standby generator rated at 150 KW.

The electrical, instrumentation and SCADA systems will be upgraded by the replacement of some existing equipment and the addition of new equipment as detailed in this Technical Memo.

Retained Systems

The following existing electrical systems will be retained:

- The 230 VAC electrical feed to the existing Primary Clarifiers. This feed will be used to power the new Fine Screen.
- The 230 VAC feed to the existing Administration / Laboratory Building.

Main Power Feed

Based on estimated load data shown in Table 15.1, the total Plant includes a demand load of 600 KVA. We recommend using a 1200A main service rated at 480/277 VAC, 3-phase. The service will include a standby diesel generator with automatic start and transfer switch that runs the Plant if normal power fails. Return to normal upon utility power restoration would be a manual operation.

Utility power enters the Plant at the existing service pole. The existing 240V transformers on the pole will be replaced with 480V transformers, rated as determined by TPUD. The main service will include utility metering, distribution circuit breakers and the automatic transfer switch. Motor control centers, variable frequency drives and a step down transformer rated at 240 VAC, 3-phase for existing facilities will be used.

The Electrical Equipment List shown in Table 15.1 presents estimated electrical load data for the expanded Tillamook WWTP.

Standby Generator

A generator rated at an estimated 400 KW is required to service all loads required for minimal Plant operation under power fail situations. The generator will be provided with fuel storage for approximately 24 hours of service. The generator will be located inside the new Blower and Standby Generator Building discussed in Technical Memo No. 13. The generator will be rated at 480VAC, 3-phase.

The existing 150 kW generator, installed as part of the 2003 CAP project, will be relocated to the 12th Street Pump Station.

Distribution and MCCs

- A new distribution board will be provided. It will be rated at 480 VAC, 3-phase, 1200A. The assembly will include the Automatic Transfer Switch for connection of the standby generator. The distribution board will include a 480/230 VAC transformer to connect to existing services.
- Two new Motor Control Centers (MCCs) will be provided. One MCC will be located at the Blower and Standby Generator Building and will serve the five influent pumps, other local process and building services, and miscellaneous new services. The other MCC will be located at the Solids Processing Building and will serve the equipment associated with this facility. The MCCs will be rated at 480 VAC, 3-phase. The MCCs will include a 480/120 VAC transformer for other loads and lighting.

Lighting

Additional flood lighting will be provided in the areas where it is required for safe operation of unit processes and at doorways of the new buildings. Lighting will be similar to existing.

Hazardous Area Classification

The area surrounding the digester will be classified as a hazardous area, in accordance with the NEC and NFPA 820. All equipment located within the hazardous area will be suitable for the location. Hazardous area drawings will be prepared as required.

Instrumentation

- The following major instruments will be included for Plant process control and monitoring:
- Influent Wet Well Level transmitters

- Influent Flow meters
- RAS and WAS Flow meters
- Peak Flow Bypass Meter
- Miscellaneous Process Meters (see TM 4)
- Aeration Inlet Air Flow meter
- Digester Gas Pressure transmitter
- Dissolved Oxygen probes in Aeration Basin
- Chlorine Residual Analyzer or ORP (reference TM 8)
- Deox/2000 Chlorine / Sulfur Dioxide Residual Analyzer (existing)
- Effluent Flowmeter
- Plant Recycle Flowmeter

Control Philosophy

This section defines general control and monitoring philosophy for the facilities. New facilities shall be automated as described herein. The control systems shall be designed such that:

- The process shall be instrumented to the extent required to allow automated, unmanned operation of the headworks screen, the aeration blowers, the RAS system, the disinfection system, and the dechlorination system.
- Process operations shall be automated, controlled by process control programs executing in local or centralized PLCs.
- In the event of PLC failure, all equipment can be operated locally by the use of manual switches.
- A SCADA system shall be implemented to provide centralized monitoring, alarm notification, and operator control, as described below.
- Operator control of the processes will normally be initiated from the SCADA system screen (human-machine interface, HMI) located in the Administration / Laboratory Building.
- The PLCs and SCADA system shall be powered from a UPS, rated to provide operation of the connected PLC's and instrumentation for four hours.
- Alarms from the 12th Street Pump Station should be integrated into the SCADA system.
- Some form of dial-up system will be integrated into the control system to notify the operation staff of alarm conditions.

Control Modes

When process equipment is provided with local and remote (hand and automatic) control capability, coordination between local control and control provided through PLCs are extremely important. Control Modes will be coordinated through the use of LOCAL-OFF-REMOTE switches.

The following control modes will be implemented:

- LOCAL: Changes to process equipment are initiated from the L-O-R switch. A pump in the LOCAL position shall be controlled via use of local START and STOP buttons. A valve may be opened and closed through the use of local OPEN and CLOSE buttons or a switch.

- OFF: All commands to the device are disabled. A pump in the OFF position will be stopped.

- REMOTE: Changes to process equipment are initiated from the PLC, either by an operator issuing manual commands, or by a control program running in the PLC.

Process Monitoring

All process parameters such as flows, densities, levels, etc. are to be displayed as required on the HMI. The status of major process equipment shall be indicated locally and transmitted to the SCADA system.

The following are general guidelines for selection of points to be monitored by the SCADA system:

- Analog values for all major process variables such as flow, level, residual disinfection chemical levels, temperature and pressure

- The status of all major process equipment such as pumps and blowers

- The status of all process variables or equipment monitored or controlled by a PLC

- The status of all Local/Off/Remote switches

- Limit switches on electrically-actuated gate on the PEFCS

- Motor speed for all variable speed devices that are to be monitored or controlled by the SCADA system

- All safety indicators (e.g. chlorine leak)
- High water level alarms in plant pump stations and the headworks

Manually-actuated valves will not be monitored.

Critical Control Equipment

- Critical control equipment will be identified based on the following criteria: a) equipment required during failure of the main power feed that must be fed from the standby generator, and b) those instruments deemed critical for continuous operation that will be provided with a level of redundancy by the installation of a second instrument permanently connected to the control system. Failure of the main power supply is assumed to be for not more than 24 hours.

- Critical Equipment during Main Power Failure
  - One Influent Jockey Pump
  - One Influent Main Pump
  - Fine Screen
  - One or Two Aeration Blowers (to be determined following final blower selection)
  - One RAS Pump (large unit)
  - One WAS Pump
  - Two Aeration Basin Mixers and one Plant Recycle Pump
  - Both Primary Clarifier Drives
  - Both Secondary Clarifier Drive
  - Chlorinator and Chlorine Mixer
  - Sodium Bisulfite Feed System and Mixer

- Critical Instruments
  - Influent wet well level
  - Dissolved oxygen probes
  - Chlorine residual analyzer or ORP
  - Deox/2000 analyzer

SCADA System

The control system will consist of Programmable Logic Controller(s) connected to a SCADA System computer and display screen. Some process systems such as the Fine Screen will be designed with a vendor-provided PLC and control panel. These packaged systems will be incorporated in to the SCADA System.

The PLCs will be connected via a data network, and all data will be transmitted to the SCADA System computer. The SCADA System computer will run an operator interface software (HMI) such as iFix or Wonderware to monitor, display and trend process data such as level, flow and water quality measurements. The HMI will provide the operator with the ability to control all major equipment, such as pumps and valves.

- The SCADA System will provide for alarm generation and notification throughout the Plant and will support a dial-out alarm capability.
- The existing mimic and alarm panel in the Administration / Laboratory Building will be de-commissioned and removed.

Recommendations for Additional Investigation During Final Design

1. The utility service pole at the site provides for an additional connection to the adjacent park. Electrical facilities at the park consist of area lighting, and lighting and air dryers in the washrooms. Additional investigation should be made to determine if the electrical service from the Plant to the utility pole serving the park could be made underground. Given the close proximity of the utility pole to the RBCs (to be demolished) and some trees (that may be removed), electrical service to the park may be disconnected for periods during construction. During the final design phase, the disconnection period will be planned and minimized.

2. Given the increased size of electrical service to the Plant, TPUD may wish to use a pad-mounted 480 VAC 3-phase transformer rather than pole-mounted units. This option can be explored with TPUD.

    - High water level alarms in the collection system pump stations may be integrated in to the SCADA System. This would require the provision of a radio-based telemetry system.

    - An alternative location for the SCADA HMI may be investigated as the Administration Building has very little available space for a new computer.

Table 15.1 - Electrical Equipment List
Tillamook WWTP Expansion

Equipment Identification			Timing		Motor Data					Misc. Data										Controls & Accessories							
Tag	Name on MCC Namplate	P&ID Drawing Number	Provision For Future	Modify Existing	Horsepower	Speed(s)	Orientation: (H)orizontal, (V)ertical	Location: (I)ndoor, (O)utdoor	Enclosure: (E)xplosion Proof, (O)pen, (S)ubmersible, (T)EFC	KW (Heating, Ozonators, etc.)	Voltage	3 Phase or 1 Phase	Full Load Amperes	Demand Factor	Demand Load KVA	On Backup Power?	Backup Demand KVA	Conductor Size	Conductor size Amps	Adjustable Speed	H-O-A Control	Manual Control	Package Starter & Controls	Provide Running Time Meter	Interlock with Other Motors	Remarks	Revision Date
	480 V Equipment																										
	Influent Pump Station																										
	Pump 1				45						460	3	52	100	41.4	Y	41.4	#6	65	Y							
	Pump 2				45						460	3	52	100	41.4	Y	41.4	#6	65	Y							
	Pump 3				45						460	3	52	0	0.0	N	0.0	#6	65	Y							
	Pump 4				18						460	3	21	100	16.7	Y	16.7	#10	27	Y							
	Pump 5				18						460	3	21	0	0.0	N	0.0	#10	27	Y							
	Scum Pump Station																										
	Chopper Pump				5						460	3	7.6	100	6.0	Y	6.0	#12	10								
	Electric Valve				0.5						460	3	1	100	0.8	Y	0.8	#12	2								
	Recycle Pump Station																										
	In-Plant PS Pump				5						460	3	7.6	100	6.0	Y	6.0	#12	10								
	Septage Receiving Station																										
	Drum Gearmotor				0.5						460	3	1	100	0.8	Y	0.8	#12	2								
	Floc Gearmotor				0.5						460	3	1	100	0.8	Y	0.8	#12	2								
	Booster Pump Motor				1.5						460	3	2.6	100	2.1	Y	2.1	#12	4								
	Aeration Basin																										
	Blower 1				60						460	3	77	100	61.3	Y	61.3	#3	97								
	Blower 2				60						460	3	77	100	61.3	Y	61.3	#3	97								
	Blower 3				60						460	3	77	100	61.3	N	0.0	#3	97								
	Blower 4		Y		60						460	3	77	100	61.3	N	0.0	#3	97								
	Blower 5		Y		60						460	3	77	0	0.0	N	0.0	#3	97								
	Submersible Mixer 1				5						460	3	7.6	100	6.0	Y	6.0	#12	10								
	Submersible Mixer 2				5						460	3	7.6	100	6.0	Y	6.0	#12	10								
	Submersible Mixer 3				5						460	3	7.6	100	6.0	N	0.0	#12	10								
	Submersible Mixer 4		Y		5						460	3	7.6	100	6.0	Y	6.0	#12	10								
	Submersible Mixer 5		Y		5						460	3	7.6	100	6.0	Y	6.0	#12	10								
	Submersible Mixer 6		Y		5						460	3	7.6	100	6.0	N	0.0	#12	10								
	Digesters																										
	Mixing Pump 1				20						460	3	27	100	21.5	Y	21.5	#10	34								
	Recycle Pump 1				2.5						460	3	3.4	100	2.7	Y	2.7	#12	5								
	Boiler Recycle Pump 1				0.5						460	3	1	100	0.8	Y	0.8	#12	2								
	Mixing Pump 2		Y		20						460	3	27	0	0.0	N	0.0	#10	34								
	Recycle Pump 2		Y		2.5						460	3	3.4	0	0.0	N	0.0	#12	5								
	Boiler Recycle Pump 2		Y		0.5						460	3	1	0	0.0	N	0.0	#12	2								

Table 15.1 - Electrical Equipment List
Tillamook WWTP Expansion

Tag	Name on MCC Namplate	P&ID Drawing Number	Provision For Future	Modify Existing	Horsepower	Speed(s)	Orientation: (H)orizontal, (V)ertical	Location: (I)ndoor, (O)utdoor	Enclosure: (E)xplosion Proof, (O)pen, (S)ubmersible, (T)EFC	KW (Heating, Ozonators, etc.)	Voltage	3 Phase or 1 Phase	Full Load Amperes	Demand Factor	Demand Load KVA	On Backup Power?	Backup Demand KVA	Conductor Size	Conductor size Amps	Adjustable Speed	H-O-A Control	Manual Control	Package Starter & Controls	Provide Running Time Meter	Interlock with Other Motors	Remarks	Revision Date
	Thickener																										
	Gravity Belt Thickener				0.8						460	3	1.4	100	1.1	Y	1.1	#12	2								
	Sludge Return Pump				5						460	3	7.6	100	6.0	Y	6.0	#12	10								
	Secondary Clarifier																										
	Clarifier 1				2						460	3	3.4	100	2.7	Y	2.7	#12	5								
	RAS Pump 1				5						460	3	7.6	0	0.0	N	0.0	#12	10								
	RAS Pump 2				15						460	3	21	100	16.7	Y	16.7	#12	27								
	WAS Pump 1				2.5						460	3	3.4	100	2.7	Y	2.7	#12	5								
	Scum Pump 1				0.5						460	3	1	100	0.8	Y	0.8	#12	2								
	Electric Valve 1				0.5						460	3	1	100	0.8	Y	0.8	#12	2								
	Clarifier 2				2						460	3	3.4	100	2.7	Y	2.7	#12	5								
	RAS Pump 3				5						460	3	7.6	0	0.0	N	0.0	#12	10								
	RAS Pump 4				15						460	3	21	100	16.7	N	0.0	#12	27								
	WAS Pump 2				2.5						460	3	3.4	100	2.7	Y	2.7	#12	5								
	Clarifier 3		Y		2						460	3	3.4	0	0.0	N	0.0	#12	5								
	RAS Pump 5		Y		10						460	3	14	0	0.0	N	0.0	#12	18								
	RAS Pump 6		Y		10						460	3	14	0	0.0	N	0.0	#12	18								
	WAS Pump 3		Y		2.5						460	3	3.4	0	0.0	N	0.0	#12	5								
	Chemical Equipment																										
	Chlorine Injection Mixer				2						460	1	3.4	100	2.7	Y	2.7	#12	5	Y							
	Sodium Bisulfite Injection Mixer				2						460	1	3.4	100	2.7	Y	2.7	#12	5	Y							
	Chlorine Sample And Eductor Pump				2						460		3.4	100	2.7	Y	2.7	#12	5								
	Miscellaneous																										
	Fine Screen (added to Panel LA)				0.8								1.4	100	1.1	Y	1.1	#12	2								
	Utility Water Pump				7.5						460	3	11	100	8.8	Y	8.8	#12	14	Y							
	230 V Equipment																										
	Existing Equipment																										
	MCC-A									59.4	230	3	0	100	59.4	Y	59.4										
	Panel LA									45	230	3	0	100	45.0	Y	45.0										
	120/240 V Equipment																										
	Typical Analyzers																										
	pH Analyzer									0.1	115	1	0	100	0.1	Y	0.1										
	Chlorine Analyzer									0.1	115	1	0	100	0.1	Y	0.1										
	Fluoride Analyzer									0.1	115	1	0	100	0.1	Y	0.1										
	Sodium Bisulfite Feeder									0.1	115	1	0	100	0.1	Y	0.1										
	HVAC / Plumbing Equipment																										
	Blower and MCC Room Exhaust fan									1	115	1	0	100	1.0	Y	1.0										

Table 15.1 - Electrical Equipment List
Tillamook WWTP Expansion

Tag	Name on MCC Namplate	P&ID Drawing Number	Provision For Future	Modify Existing	Horsepower	Speed(s)	Orientation: (H)orizontal, (V)ertical	Location: (I)ndoor, (O)utdoor	Enclosure: (E)xplosion Proof, (O)pen, (S)ubmersible, (T)EFC	KW (Heating, Ozonators, etc.)	Voltage	3 Phase or 1 Phase	Full Load Amperes	Demand Factor	Demand Load KVA	On Backup Power?	Backup Demand KVA	Conductor Size	Conductor size Amps	Adjustable Speed	H-O-A Control	Manual Control	Package Starter & Controls	Provide Running Time Meter	Interlock with Other Motors	Remarks	Revision Date
	Solids Handling Building Exhaust Fan									1	115	1	0	100	1.0	Y	1.0										
	Blower and MCC Room Heater									5	208	1	0	100	5.0	Y	5.0										
	Solids Handling Building Heater									5	208	1	0	100	5.0	Y	5.0										
	Control Room Equipment																										
	MCP									1.5	115	1		100	1.5	Y	1.5										
	SCADA System									1	115	1		100	1.0	Y	1.0										
	Fire Alarm System									0.2	115	1		100	0.2	Y	0.2										
	Security System									1	115	1		100	1.0	Y	1.0										
	Lighting																										
	Fluorescent fixtures							24		150	115	1		100	3.6	Y	3.6										
	Exterior lighting							4		100	115	1		100	0.4	Y	0.4										
	Yard lighting							3		150	115	1		100	0.5	Y	0.5										
	Halide Bay lighting							0		175	115	1		100	0.0	Y	0.0										
	DEMAND LOAD KVA														**618.3**		**467.0**										
	DEMAND LOAD AMPS														**744.6**		**562.4**										
	GENERATOR KW (approx)																**396.9**										

Technical Memorandum No. 16

Subject: Civil Work, Landscaping and Site Security
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Harry Ritter, P.E.

Introduction

The majority of the construction required for the WWTP upgrade will be undertaken on the "North Site" property. This property, located adjacent to the existing site, is bordered by 3rd Street to the north, the Trask River to the west, single family and multi-unit residential units to the east, and the existing WWTP to the south. The site is relatively flat, dropping approximately two feet in grade from east to west. The river bank is short and steep falling about six feet to the average water surface. The proposed site layout would accommodate a future plant expansion which could potentially double the capacity of the upgraded facility. The proposed layout is shown on the Proposed Site Plan, Figure 16.1.

A geotechnical investigation of the North Site was undertaken in September 2005. The investigation found that the site is defined by two distinct soil layers. The upper layer, or mantle, is silt material consisting of alluvial flood deposits, and extends to depths of 6.5 to 7.5 ft. This layer has bearing capacity of approximately 2,500 psf. The mantle layer is underlain by interbedded sequences of layered gravel, sand, and silt that likely represent severely weathered sedimentary rock. This material is classified as being very stiff to hard and extends to the maximum depth explored. Groundwater levels at this site are expected to closely parallel river levels. An additional geotechnical investigation will be conducted on 17 January 2006 in order to determine if the southwest corner of the site is suitable for construction of the anaerobic digester and Solids Processing Building.

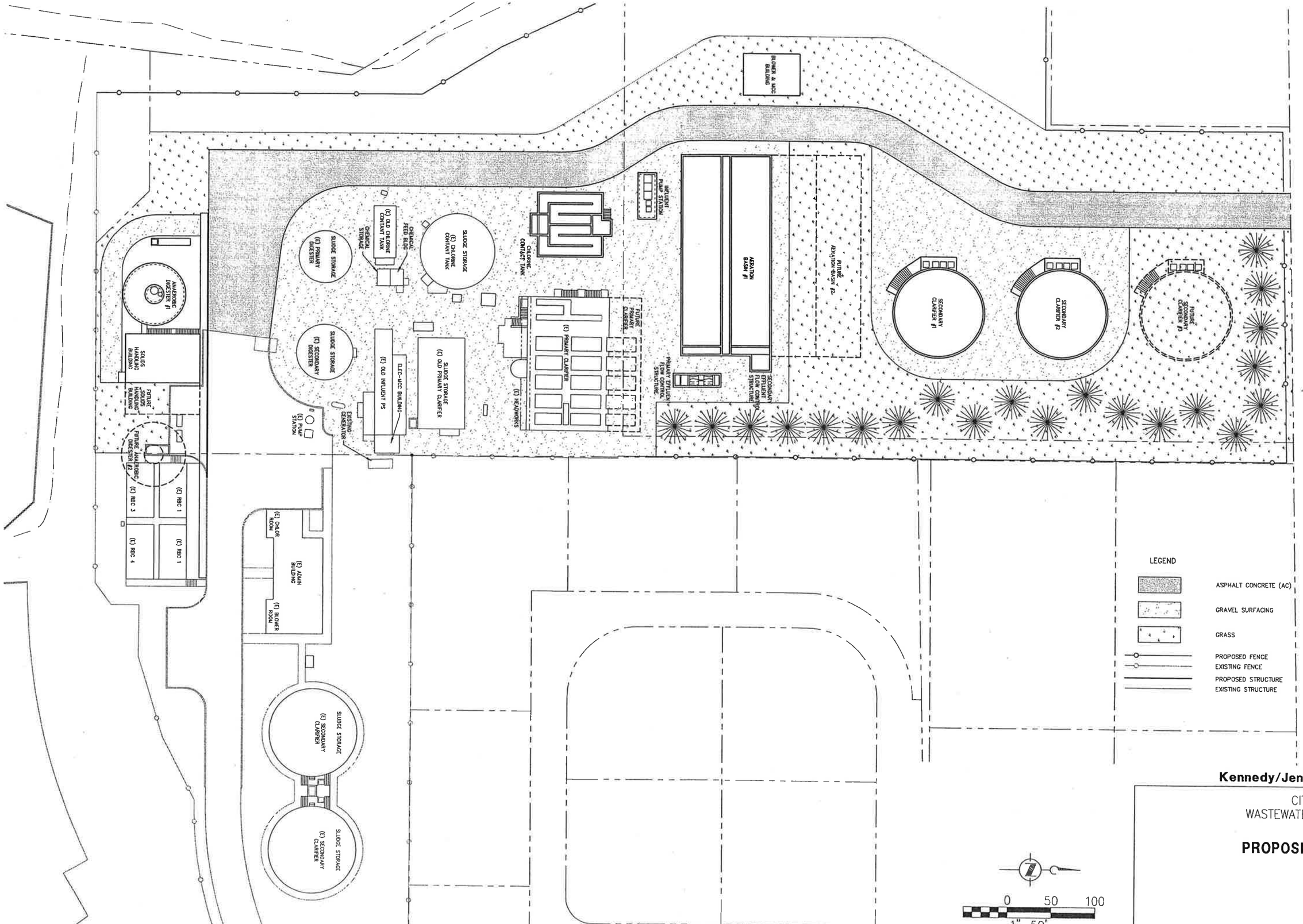
Top soil from the site was tested for organic pesticides as the field has been used for agricultural purposes in the past. The tests, conducted by a certified laboratory, indicate that all of the organic pesticides listed under EPAs Method 8081 are below detectable levels.

In general, the site is well suited for the proposed construction although the presence of groundwater will necessitate dewatering during construction. The Geotechnical Report is included as a separate section of this report.

Civil Site Work

Demolition

Budget for demolition of the existing RBCs, the existing raw influent screens, and the existing influent pump station has been included in the preliminary cost estimate. The RBC basins are not properly configured for conversion to sludge storage structures or for other practical uses and the area they occupy may become useful for future treatment plant expansion. The existing headworks screens are dilapidated and should be removed. There may be value in converting the existing influent pump station building into equipment storage or workshop space. The City



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CITY OF TILLAMOOK
WASTEWATER FACILITY PLAN

PROPOSED SITE PLAN

0491010.20

FIGURE 16.1

will make the final determination. Since these structures are still in use, their demolition will need to be undertaken following startup of the new facility.

Excavation

The amount of fill and barrow material required for construction activities will depend on the means and methods of the excavation contractor. Imported structural backfill material will be required as the native material is not expected to re-compact well following excavation. Excavated material will need to be hauled offsite.

While the Blower Building and the Solids Processing Building will be constructed at-grade, the new treatment structures will have their foundations set below the alluvial silt mantle on the weathered sedimentary rock. Table 16.1 shows the approximate burial depths for the proposed structures.

Table 16.1 – New Structure Elevations

New Structure	Top of Structure Elevation (ft)	Floor of Structure Elevation (ft)	Finish Grade (ft)	Burial Depth (ft)	Above Grade Height
Influent Pump Station	14	-6	14	20	0
Flow Control Structure	26	14	14	0	12
Aeration Basin	25	5	14	9	11
Secondary Clarifier Flow Splitter	25	14	14	0	11
South Secondary Clarifier	21	6	14	8	7
North Secondary Clarifier	21	6	14	8	7
Anaerobic Digester	28	0	14	14	14
Chlorine Contact Tank	17	5	14	9	3
Solids Processing Building	30	14	14	0	16
Blower Building	28	14	14	0	14

Stormwater Permitting and Impervious Surface Runoff

As of 1 December 2002 the Oregon DEQ requires a National Pollutant Discharge Elimination System (NPDES) 1200-C general stormwater permit for any construction activity disturbing one acre or more of land. The WWTP construction will fall under that requirement. To obtain the permit, the applicant must submit a 1200-C permit application and a Land Use Compatibility Statement to DEQ before starting construction. The application will require an Erosion and Sediment Control Plan and a fee of $670. The Erosion and Sediment Control Plan will be

incorporated in the WWTP Design Drawings. The permit application should be undertaken in conjunction with the final design effort.

Stormwater runoff from impervious surfaces generated by the expanded facility will be retained onsite. Paved areas will be sloped to allow stormwater to sheet flow to the nearest edge of pavement. Grassy swells along paved areas will provide for the treatment and percolation of pavement runoff. Stormwater runoff from rooftops will be piped to an onsite retention pond for treatment and eventual percolation.

All of the chemicals used at the treatment plant will be stored indoors eliminating the opportunity for chemical contamination of stormwater runoff.

Paving and Grading

The extent of the proposed paving is shown on Figure 16.1. The base preparation and AC pavement should be in accordance with APWA specifications and designed to serve the type of truck traffic anticipated at the WWTP.

Landscaping

New landscaping at the WWTP will consist of establishing a smooth surface of hardy turf grass in un-paved areas and a screening hedge along the fence line on the east and north sides of the property. The hedge is intended to visually screen the WWTP from its neighbors and from traffic along 3rd Street. Shrubs used for the hedge should be low maintenance and provide year-round cover. Consideration should be given to the installation of an irrigation system using treated effluent to help establish and maintain the hedge and other landscaping. Some of the existing pine trees may be removed from the site to reduce seasonal maintenance.

Site Security

Site security will likely consist of full-perimeter fencing, electrically actuated access gates, outdoor security cameras, motion detectors and area lights, and dry contact switches on all building doors and windows. The intent of the security system is to deter theft and vandalism and to provide a means to alert authorities to an unauthorized entry. The security system will be integrated with the SCADA system for remote notification of intrusion or alarm. A discussion between the security designer and the Owner will be scheduled early in the design process to establish the minimum requirements of the security design.

Approximately 1,300 feet of new chain-link fence will be required to surround the new site. The new fence should match the existing fence in height and materials of construction. A screen fence that provides both security and an attractive appearance could be considered to in lieu of providing a screening hedge along the chain-link fence.

Technical Memorandum No. 17

Subject: Front Street Sodium Hypochlorite Dosing Station
City of Tillamook WWTP Expansion
K/J No. 0491010.10

Submitted By: Tom Leavitt, P.E. and Harry Ritter, P.E.

Background

Hydrogen sulfide gas, released when the STEP system pressure main discharges to the manhole at the intersection of 1st Street and Stillwell Avenue, is a source of odor and causes corrosion of the manhole and downstream pipes. As a result, the manhole and a 20-foot length of pipe have eroded to the point that replacement is required. The manhole and downstream piping are currently scheduled to be replaced as part of the Phase One Sewer Improvements. The new manhole will be coated with a corrosive resistant epoxy coating and the existing clay pipe will be replaced with PVC pipe to prevent future corrosion.

H2S Control

Dosing with sodium hypochlorite is the most common method for hydrogen sulfide gas control and is the method selected to control formation of H2S at the 1st Street and Stillwell Avenue manhole. Sodium hypochlorite can be purchased in bulk strength solutions, normally at 12.5 percent concentration. Bulk delivery has many advantages such as ease of operation and maintenance, flexibility, reliability, and reduced cost.

Chlorine Demand

Previous testing by City personnel has found that the H2S concentrations in the wastewater entering the manhole are normally between 1 mg/l and 10 mg/l. Typically, it takes 5:1 chlorine to H2S concentration to oxidize and provide effective odor control in sewage as there are other compounds present creating a chlorine demand. Table 17.1 indicates the expected chlorine requirements to treat the 1st Street and Stillwell Avenue manhole.

Table 17.1 - Chlorine Demand

	Units	Maximum [2]	Average [1]
Wastewater Flows	gpm	220	22
Average H2S Concentration	mg/l	5.0	5.0
Chlorine Feed Rate	lb/day	66	6.6
Feed Pump Requirement	gph	2.8	0.28
Chlorine Daily Usage [3]	gpd	na	6.7

[1] based on 62 STEP systems contributing 500 gallons per day (gpd).
[2] based on peaking factor of 10.
[3] based on sodium hypochlorite concentration of 12.5%

Bulk Sodium Hypochlorite

Typically, bulk sodium hypochlorite is delivered as trade solution of 12.5 percent, which contains approximately 1.04 lbs of available chlorine per gallon of solution.

12.5 percent sodium hypochlorite tends to degrade fairly rapidly. Its stability is affected by the following factors:

- Concentration of the hypochlorite solution (higher concentration solutions, such as 12.5 percent trade strength, break down much more rapidly than low strength solutions.)
- pH (high pH stabilizes the solution)
- Temperature (degradation increases with increasing temperatures)
- Presence of heavy metal cations (in descending order: nickel, cobalt, copper, iron, manganese, mercury, aluminum, lead, zinc, tin, magnesium, and barium) tends to increase degradation and increase gas production. It is, therefore, important to specify a refined or filtered grade of chemical to reduce the concentration of these by-product constituents.
- Exposure to UV light will increase the rate of degradation.

Limiting storage of sodium hypochlorite to 60 days due to its continuing loss of available chlorine is normally recommended. A 45-day delivery schedule will reduce the overall degradation and be more economical in terms of chemical use. However, the delivery supplier may discount for larger volumes of chemical so the loss of chemical strength with storage time must be weighed against discounted delivery costs for larger volumes. It is recommended that a storage tank with a 90-day storage volume be provided for this project. This would require a tank volume of approximately 600 gallons. It is likely that tanks will commonly be available in 500 gallon or 750 gallon sizes. If this is the case, a 750-gallon tank is recommended.

Hypochlorite solutions (12 to 15 percent by weight) are classified as a corrosive material under the health hazard category and must be handled with care.

System Layout

It is anticipated that the chemical storage and dosing equipment would be located on the existing Front Street Pump Station property. According to the City's tax map, this property is designated as City of Tillamook right-of-way. The sodium hypochlorite will be injected into the force main at this location to provide adequate time for mixing prior to discharge to the manhole at 1st Street and Stillwell Avenue.

Sodium hypochlorite solution must be stored with a dual-containment safeguard. A dual-containment system can be provided by using a dual-wall tank or by constructing a cast-in-place or CMU containment structure with sufficient volume to contain the storage tank contents. A dual-wall tank is assumed for this project as it is typically less-expensive. Some chemical suppliers will provide a dual-wall tank free of charge in exchange for a long-term supply contract. This potential arrangement should be investigated during design. The tank should be located so that it can be easily accessed by the delivery truck.

The two existing 4-inch forcemains from the STEP system in Sub Basin 5 will be intercepted north of Front Street, joined together with a 4-inch tee, and routed through the flowmeter/injection vault as shown schematically on the Front Street Sodium Hypochlorite Dosing Station Schematic, Figure 17.1. The hypochlorite solution will be injected into the forcemain using a chemical metering pump. A solenoid-type metering pump, capable of receiving a 4-20 mA flow signal, should be adequate to perform the required service at the lowest price point. The metering pump will be flow-paced based on the flowmeter readings. The meter will be a mag meter suitable for the intended service. The chemical injection quill should be located downstream of the meter. The metering pump and meter electronics will be located in a corrosive resistant fiberglass reinforced plastic (FRP) shelter. Consideration should be given to preventing vapor lock in the delivery system as sodium hypochlorite will off-gas and air-lock pumps and lines. The shelter will be equipped with lights, a small heater, and an exhaust fan. Feed system appurtenances are shown on Figure 17.1.

Miscellaneous Design Details

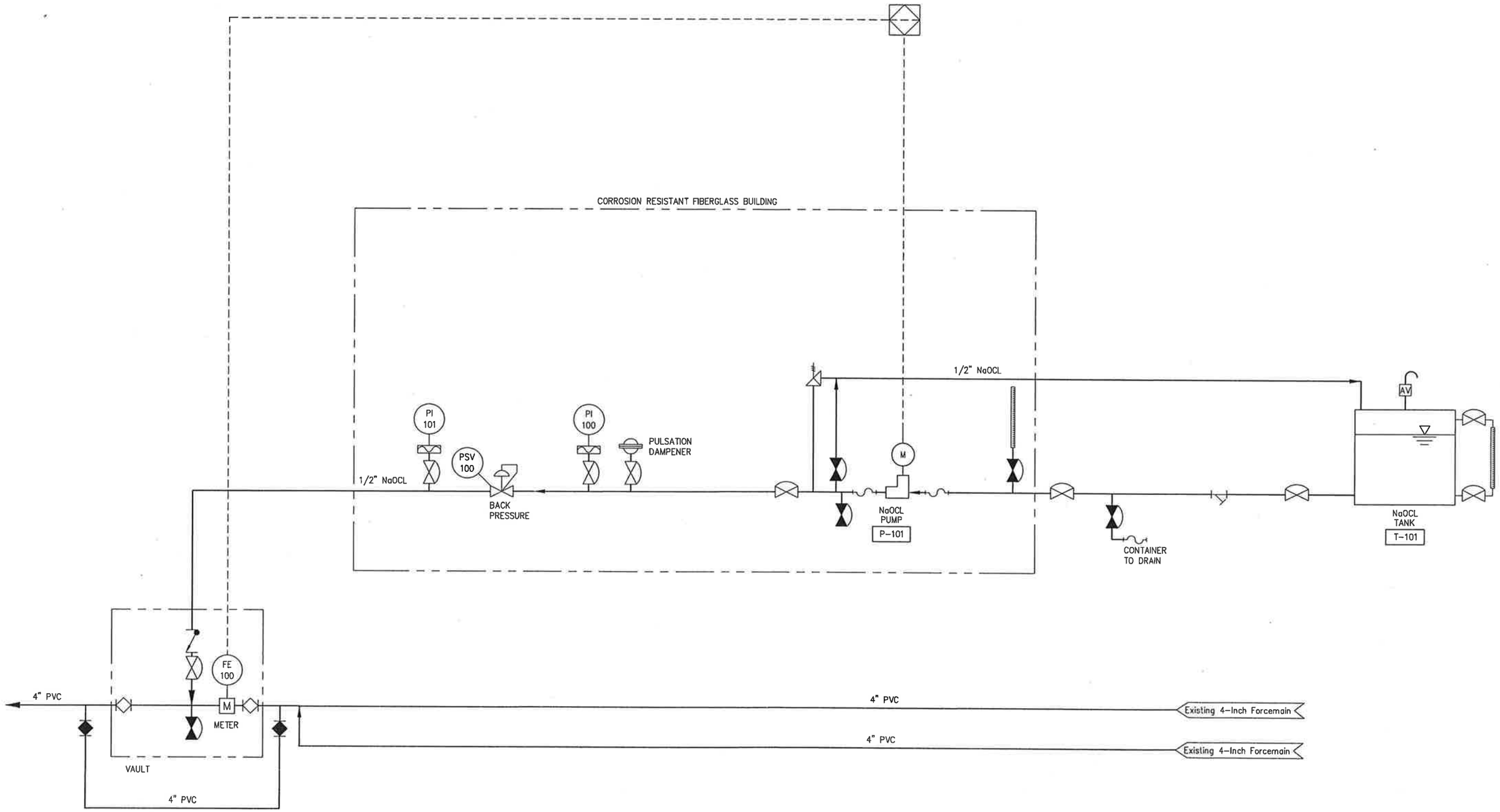
A concrete pad will be poured to install the FRP feed pump/meter building and to locate the chemical storage tank. The FRP building should be bolted to the slab and be capable of withstanding the wind loading and seismic forces required by local Building Code. The FRP building should be equipped with a lockable door and padlock. An 8-foot chain-link fence with swinging gate will be installed to allow for easy personal and chemical truck access to the site.

Storage tanks are generally constructed of either FRP or HDPE. Both tanks offer similar advantages and the final selection should be based on lifespan costs. If FRP is used, the resin and veil coat should be specifically suited for the storage of sodium hypochlorite. The storage tank material should be highly pigmented to block ultra violet radiation.

The chemical feed line from the tank to the building should be heat traced. The building should have lights and an electric heater to maintain temperatures above freezing. Diaphragm valves should be used for all chemical service lines. Plug valves should be used for all sewage service lines.

A sight glass should be specified to measure the liquid level in the storage tank. The fill station connections would be equipped with keyed locking caps.

The meter/injection vault will be fitted with an H-20 rated aluminum hatch cover. An exterior light capable of illuminating the site should be provided.



Kennedy/Jenks Consultants

City of Tillamook, Oregon

Front Street Sodium Hypochlorite Dosing Station Schematic

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Figure 17.1

P:\cad\04\0491010.20_WW 2N\Final Predesign Figures\Fig 17-1 Front St Dosing Sta Sch.dwg Nov 28, 2005 - 11:57am

Local fire codes should be checked for requirements.

Chemical Costs

Currently, the cost of 12.5 percent sodium hypochlorite delivered by Northstar Chemical in Portland is $1.19/gal for 5,000 gallons, $1.50/gal for 2,000 gallons, and $1.82/gal for 500 gallons.

GRI Geotechnical & Environmental Consultants
9725 SW Beaverton-Hillsdale Hwy, Ste 140

November 15, 2005

4144 GEOTECHNICAL RPT

Kennedy/Jenks Consultants
200 SW Market St. Suite 500
Portland, OR 97201

Attention: Harry E. Ritter, PE

SUBJECT: Geotechnical Investigation
Wastewater Treatment Plant Expansion Project
Tillamook, Oregon

At your request, GRI has completed a geotechnical investigation for the above-referenced project. The Vicinity Map, Figure 1, shows the approximate location of the site for the wastewater treatment plant expansion in Tillamook, Oregon. Our investigation consisted of subsurface explorations, laboratory testing, and engineering analyses. This report summarizes our findings and presents our conclusions and recommendations regarding the geotechnical aspects of the design and construction of the proposed plant additions and improvements.

PROJECT AND SITE DESCRIPTION

General

The expansion site is located on the south side of 3rd Street just to the east of the Trask River and north of the existing treatment plant in Tillamook. The site is presently used as a pasture and existing grades are relatively flat at about elevation +14 ft. The Site Plan, Figure 2, shows the layout of the proposed expansion project. As you know, the original expansion plans were located east and west of the existing treatment plant. Information you provided regarding the dimensions of the new structures is summarized below.

BASIC DIMENSIONS OF PROPOSED STRUCTURES

Structure	Plan Dimensions, ft	Inside Depth Below Grade, ft	Height of Wall Above Grade, ft	Normal Fluid Depth, ft
Influent Pump Station	26.5 by 22	20	At grade	4
Effluent Flow Control Structure	14 by 11	12	12	10.5
Aeration Basin	95 by 47	20	11	18
Secondary Clarifier Flow Splitter	12 by 10	6	7	4.5
South Secondary Clarifier	65 diameter	15	7	13
North Secondary Clarifier	65 diameter	15	7	13
Anaerobic Digester	30 diameter	20	6	20
Chlorine Contact Tank	37 by 37	12	3	10
Solids Handling Building	32 by 32	At grade	20	N/A
Blower Building	32 by 17	At grade	10	N/A

Topography and Geology

The ground surface in the surrounding area is relatively flat except where it slopes downward along the bank of the river. Detailed topographic information for the area is not yet available for review; however, as previously indicated, we understand the existing ground surface in the expansion area is on the order of elevation +14 ft. Available geologic information for the area indicates the site is mantled with silt, likely representing overbank flood deposits of the adjacent river, and underlain by layered silt, sand, and gravel that may represent severely weathered sedimentary rock.

SUBSURFACE CONDITIONS

General

Six borings, designated B-1 through B-6, were drilled on September 28 and 29, 2005, to evaluate subsurface materials and conditions at the site. The borings were advanced to depths of 20.5 to 35.8 ft at the approximate locations shown on Figure 2, which also shows the location of a boring (B-4) made during our geotechnical investigation for the original expansion plans in September 2004. Appendix A provides a detailed discussion of the field exploration and laboratory testing programs competed for this investigation. Logs of the borings are provided on Figures 1A through 7A. The terms used to describe the soils encountered in the explorations are defined in Tables 1A and 2A.

Soils

For the purpose of discussion, the soils disclosed by the borings have been grouped into the following categories:

1. **Alluvial SILT**
2. **Layered GRAVEL, SAND, and SILT (severely weathered sedimentary rock)**

1. Alluvial SILT. Silt, considered to represent relatively alluvial flood deposits, mantles the ground surface at all the boring locations and extends to depths of 6.5 to 7.5 ft. A heavily rooted zone extends to a depth of about 4 in. The silt ranges from light to dark brown and typically contains a trace to some clay and sand. N-values of 3 to 12 blows per foot indicate the relative consistency of the silt ranges from soft to stiff and is generally soft to medium stiff. The natural moisture content of the soil ranges from about 60 to 80%, which indicates the silt has a relatively high organic content.

2. Layered GRAVEL, SAND, and SILT. The site is underlain by interbedded sequences of layered gravel, sand, and silt that likely represent severely weathered sedimentary rock; the materials extended to the maximum depth explored. The gravel is fine to coarse grained, typically subrounded to rounded, and has a matrix of silty sand. The sand is fine to coarse grained and contains varying percentages of gravel and silt. The silt is typically interbedded with thinner layers of sand and contains scattered gravel. Typical N-values ranging from about 66 blows/ft to refusal, defined as 50 blows producing less that 6 in. of sampler penetration, indicate the relative density of the sand and gravel is very dense. However, it is our experience that the Standard Penetration Test tends to overestimate the relative density of gravel. The relative consistency of the silt is very stiff to hard based on N-values of 22 blows/ft to refusal.

Groundwater

A groundwater observation standpipe was installed in borings B-2 and B-3 upon completion of drilling. The groundwater level measured in boring B-2 on September 29, the day after the standpipe was installed, was at a depth of 7.6 ft; the groundwater level has not yet been measured in boring B-3. Groundwater levels at this site are expected to closely parallel river levels and may rise to near the ground surface during periods of high water or prolonged heavy precipitation.

CONCLUSIONS AND RECOMMENDATIONS

General

The recent borings indicate subsurface conditions at the site are generally consistent with earlier borings drilled for the existing treatment plant. In general, the site is mantled with soft to stiff alluvial silt to a depth of about 6.5 to 7.5 ft. Beneath the silt are layers of very dense sand and gravel and/or very stiff to hard silt, which is likely severely weathered sedimentary rock. Groundwater levels at the site are expected to closely parallel the stages of the adjacent Trask River and could be at or near the ground surface during times of prolonged precipitation and high river levels.

The primary geotechnical-related issues associated with the planned expansion is the relatively high groundwater level at this site and the relatively deep excavations that will be required to embed the various new structures. The following sections of this report provide our conclusions and recommendations concerning the geotechnical aspects of the design and construction of the treatment plant expansion.

Site Preparation

The ground surface within areas that will receive fill or improvements such as pavements, buildings, or other structures established at grade should be stripped of existing vegetation and the heavily rooted zone. In grassy areas, stripping should be generally accomplished to a depth of about 4 in.; deeper grubbing would be required to remove heavier brush or tree roots. The strippings should be either disposed of off-site or stockpiled for future use in landscaped areas only. Following stripping or excavation to subgrade level within building or pavement areas, the geotechnical engineer should evaluate the exposed subgrade. Proof rolling with a loaded 10 yd^3 dump truck may be part of the evaluation. Soft areas detected should be overexcavated and replaced with structural fill. During and following stripping and excavation, the contractor must use care to protect the subgrade from disturbance by construction traffic.

The silt soils that mantle much of the project area are sensitive to moisture, as are the underlying gravels and sands due to their relatively high silt content. The soils above the water table will dry slowly during dry weather, and the moisture content of soils below the water table will remain above optimum for compaction year-around. For these reasons, working blankets or haul roads constructed of imported granular material will be required to provide construction access. Relatively clean (i.e., less than about 5% passing the No. 200 sieve, washed analysis), coarse-graded crushed rock, such as 4-in.-minus crushed rock, capped with a leveling course of relatively clean finer-graded rock, such as $^3/_4$-in.-minus, works well for this

purpose. The total required thickness of granular material is typically 18 to 24 in. for heavy construction traffic and 6 to 12 in. for areas of light construction activities subject to limited truck or equipment traffic. The use of a geotextile fabric between the granular material and the underlying subgrade will minimize the thickness of rock fill and tend to reduce maintenance of the haul roads during construction.

Excavation and Shoring

General. The method of excavation and support of excavation sidewalls are the responsibility of the contractor and subject to applicable local, state, and federal safety regulations regarding excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the responsibility of the contractor. The information provided below is for use by the owner and engineer and should not be interpreted to mean that GRI is assuming responsibility for the contractor's actions or site safety.

Several of the improvements will require excavations on the order of 20 to 25 ft below existing site grades. Our experience indicates an open cut is the most practical approach to accomplishing these excavations. Excavations to achieve foundation grades will encounter fine-grained soils (silt) over severely weathered sedimentary rock. The rock is severely weathered and decomposed and generally consists of silt, sand, and gravel, and has the density/consistency of a very stiff to hard or very dense soil. While it is likely these materials can be excavated economically with large hydraulic excavators, it should be anticipated that progress may be slowed due to the presence of hard silt and/or very dense sand and gravel materials.

Based on our evaluation of the materials disclosed in the borings, the short-term stability of temporary cut slopes of 1H:1V should be adequate. However, this assumes the slopes are dewatered; surface drainage is controlled so that runoff does not flow over the top of the slope and into the excavation; and surcharge loads due to construction traffic, material laydown, excavation spoils, etc., are not allowed within 10 ft of the top of the cut. In this regard, we recommend the use of fencing or barricades along the top of the cut to prevent this area from being subjected to any significant surcharge loads.

It must be emphasized that following the above recommendations will not guarantee that failure of the temporary cut slopes will not occur; however, the recommendations should reduce the risk of a major slope failure to an acceptable level. It should be realized, however, that blocks of ground and/or localized slumps in the excavation slopes may tend to move into the excavation during the construction. In our opinion, this is most likely to result if seepage occurs on the cut slopes and/or if the cut slope exposes relatively thick deposits of poorly graded granular soils, which have the potential to run into the excavation. Besides the obvious safety considerations, running soils would likely result in loss of ground which could cause significant damage to existing utilities and any adjacent structures.

Construction Dewatering. In our opinion, the contractor's management of groundwater within excavations will be a major consideration. We anticipate that groundwater encountered within excavations will result in seepage, possibly running soil conditions, and unstable

excavation sides and bottom. These conditions will require dewatering of excavations. The amount of groundwater observed in the excavations will depend on the overall permeability of materials encountered and the depth of the excavation below the surface of the groundwater. Minor amounts of groundwater inflow in shallow excavations can likely be managed by sump pumping; however, heavier flows may be encountered in the deeper excavations that will likely require the use of dewatering wells that extend below the depth of excavation. In our opinion, potential problems associated with construction dewatering can be minimized if construction is scheduled for the time of year when river and groundwater levels are lowest. However, if the excavation phase of the project extends through the wet winter and spring months, any dewatering system must be designed to manage the higher groundwater levels that may occur during periods of intense and/or prolonged rainfall and high water or flooding of the adjacent Trask River.

The contractor's dewatering system should be designed to maintain groundwater levels to at least 1 ft below the bottom of the excavation. In our opinion, the dewatering system should be designed by a professional engineer experienced in the design of dewatering systems and registered in the State of Oregon.

Shielding or Shoring. We anticipate that a limited amount of shoring, if any, will be used for this project. However, if the potential for loss of ground or lateral movement cannot be tolerated due to the close proximity of any existing structures or utilities adjacent to the excavations, the contractor will need to select an excavation support system that will limit the risk of significant lateral movement.

We anticipate that temporary shoring with up to 15 ft of exposed face may be necessary to protect existing structures or utilities adjacent to the excavations. We further anticipate that for the subsurface conditions existing at this site, a conventional, cantilevered soldier pile-and-lagging wall will be most likely used for this purpose. In this regard, due to the very dense sand and gravel layer (decomposed sedimentary rock), it will likely not be practical to drive the solider piles to the required penetration. Therefore, it should be anticipated that the soldier piles will need to be installed in drilled holes and backfilled with lean concrete. Guidelines for determining design pressures with respect to wall height for two cases for these types of structures are provided on Figure 3. Additional horizontal pressures due to surcharge loads may be estimated as 200 psf as noted on Figure 3 or by the methods shown on Figure 4.

For installation of new underground conduits associated with the improvements, we anticipate that a conventional trench shield may be used to support excavation sidewalls, provided some loss of ground or lateral movement can be tolerated in the area surrounding the excavation.

Excavation Bottom Stabilization

In deeper excavations or trenches where the subgrade may tend to loosen or soften with upward groundwater seepage, it will likely be necessary to overexcavate the bottom of the excavation to permit installation of a granular working blanket to control bottom instability and facilitate sump pumping to supplement primary dewatering. Evaluation of these conditions will

need to be made at the time of construction; however, the required thickness of the stabilization or granular working blanket may be 1 to 2 ft, depending on the effectiveness of the contractor's primary dewatering efforts and the exposed conditions. Relatively clean material, such as crushed rock with a maximum size of about 4-in. and with less than 2% passing the No. 200 sieve (washed analysis), should be used for this purpose.

Bedding and Backfilling

We recommend the base of embedded structures and any new conduits be underlain by a minimum 6-in. thickness of good-quality bedding material to serve as a leveling course. We recommend the bedding material consist of clean (not more than about 5% passing the No. 200 sieve, washed analysis), $^3/_4$-in.-minus crushed rock. It is important the bedding be underlain by firm, relatively undisturbed soils or a stabilization blanket. The bottom of the excavations should be thoroughly cleaned to remove loose soils and any soft, disturbed, or unstable areas should be overexcavated and stabilized as previously recommended.

In our opinion, due to the wet and/or silty nature of the excavated materials, we anticipate the excavated soils will be difficult to replace and compact properly without excessive handling. Therefore, excavations in improved areas, such as slabs at grade or pavement areas, should be backfilled with imported granular fill. The compacted granular fill material should consist of sand, sand and gravel, or crushed rock having a maximum size of up to $1^1/_2$ in. and not more than about 5% passing the No. 200 sieve (washed analysis). Granular backfill should be placed in lifts and compacted with vibratory equipment. We recommend compacting the backfill to 95% of the maximum dry density as determined by ASTM D 698 in the upper 4 ft of the backfill and at least 92% of this density below 4 ft. Appropriate lift thicknesses will depend on the type of material and compaction equipment used. For example, if hand-operated vibratory equipment is used, lift thicknesses should be limited to 6 to 8 in. If backhoe-mounted vibratory plates are used, greater lift thicknesses will likely be acceptable, provided the recommended densities are achieved. Backfilling procedures should be evaluated on the basis of field density test results at the time of construction. Flooding or jetting the backfill with water should not be permitted.

In unimproved areas, the excavated soils may be used to backfill trenches where a soft backfill and future settlement can be tolerated at the ground surface. The backfill at the top of a trench can be mounded to allow some settlement to occur prior to finishing the grading at the ground surface.

Overcompaction of the backfill behind the walls of embedded structures should be avoided. Overcompaction of backfill could significantly increase the lateral earth pressure on these walls.

Settlement

In our opinion, backfilling all excavations in improved areas with compacted granular structural fill as previously recommended would minimize post-construction settlement of the ground surface. However, inadequate removal of disturbed, soft, or loosened materials from the bottom of excavations may result in post-construction settlement of embedded structures

or new conduits. Bottom instability, foot traffic, or other construction activities could cause subgrade disturbance. Additional measures to minimize softening of the excavation bottoms and settlement would involve careful construction practices, such as preventing water from ponding in the bottom of excavations and completing excavation, placement of the embedded structures and conduits, and backfilling in the closest practical sequence.

Lateral and Uplift Pressures on Embedded Structures

In our opinion, it should be assumed the groundwater table could rise to the ground surface at some time during the design life of the facility. Design lateral earth pressures depend on the drainage condition provided outside the walls of an embedded structure and the ability of the wall to yield. We assume the new structures will have relatively rigid walls and will be designed to be watertight. Consequently, we recommend embedded walls be designed using a hydrostatic pressure based on an equivalent fluid having a unit weight of 90 pcf. Additional lateral loads due to surcharge pressures may be estimated using the methods on Figure 4. To account for seismic loading, the recommended lateral earth pressures (soil and surcharge) should be increased about 40%. The resultant force (seismic load only) acts a distance of 0.6H measured up from the base of the wall, where H equals the overall height of the wall.

We also recommend designing the floors of embedded structures to resist full hydrostatic uplift pressures imposed by a groundwater level at the ground surface. Common methods used to resist the net uplift force include increasing the thickness of the base slab or extending the base slab beyond the sidewall of the structure. The effective weight of the submerged backfill should be evaluated using a buoyant weight of 60 pcf, assuming embedded structures will be backfilled with granular structural fill material. Only the compacted backfill included in the vertical plane at the outside edge of the slab should be considered as additional load to resist the uplift force. Shearing stresses in the soil above the outer perimeter of the base slab should provide some additional uplift resistance; however, for the purpose of design, it is recommended that the shearing resistance of the backfill be neglected.

Shallow Foundations

Actual foundation loads for the new improvements that will be established at grade are not yet known. However, based on our previous experience with other similar projects we anticipate that these improvements will be relatively lightly loaded. For the design of these foundations, we recommend using an allowable bearing pressure of 2,500 psf for the design of spread footings. This value applies to the total of dead load plus permanently and/or frequently applied live loads, and could be increased by one-third for the total of all loads, such as wind or seismic.

We recommend placing a 4-in.-thick leveling course of granular structural fill over all shallow footing subgrades to protect the exposed soils from foot traffic and wet weather. All footings should be embedded a minimum of 2 ft below the lowest adjacent finished grade. We anticipate settlements will occur rapidly as loads are placed on foundations. Settlement due to dead load is expected to occur during construction and over a period of weeks. Live loads of short duration, such as wind or seismic forces, are not expected to produce appreciable settlement. Total and differential settlements will be a function of actual foundation loads;

however, for loads of less than 50 kips, total and differential settlement are expected to be less than 1 and 1/2 in., respectively, for adjacent and similarly loaded footings, assuming foundation preparation is accomplished as previously recommended.

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of spread footings and the underlying soil. The total shearing resistance between the foundation footprint and the soil should be taken as the normal force, i.e., the sum of all vertical forces (dead load plus real live load) times the coefficient of friction between the soil and the base of the footing. We recommend an ultimate value of 0.35 for the coefficient of friction. If additional lateral resistance is required, passive earth pressures against embedded footings or walls can be computed using a hydrostatic pressure based on an equivalent fluid with a unit weight of 200 pcf. This design passive earth pressure would be effective only if granular structural fill is used for the backfill.

Uplift forces can be resisted by the total weight of the footing and the weight of any backfill placed over the top of the footing. A bulk unit weight of 125 pcf may be assumed for granular structural fill placed over footings. However, this assumes that groundwater levels will not be allowed to rise above the base of the footing. It may be appropriate to include foundation drains to eliminate this possibility. Alternatively, a buoyant unit weight of 63 pcf should be assumed.

Seismic Design Criteria

Along the entire coast of the Pacific Northwest, the proximity of the offshore Cascadia Subduction Zone predominates the potential seismicity of the area. The 2004 Oregon Structural Specialty Code (OSSC) adopted the 2003 International Building Code (IBC) which takes this geology into consideration for the purposes of design. Regarding the assignment of IBC site classifications, IBC Site Class C for very dense soil and rock is most applicable for the site based on the subsurface conditions disclosed in the borings. For OSSC design purposes at this site, the mapped maximum considered earthquake spectral response accelerations at short periods (S_S) and at 1-second period (S_1) are 1.54 and 0.72 g, respectively. Furthermore, we anticipate there is a low potential for earthquake-induced liquefaction, lateral spreading, and settlement. However, due to the proximity of the area to Tillamook Bay and the Pacific Ocean, we anticipate there may be a risk of damage by tsunamis. The site is located in a tsunami inundation zone (Priest, 1995). However, it would likely not be economically feasible to mitigate this possible risk in the design and construction of the new improvements.

Design Review and Construction Services

GRI should be retained to review the geotechnical-related portions of the final plans and specifications to evaluate whether they are in conformance with the recommendations provided in this report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we recommend that construction operations dealing with earthwork and foundations be observed on an intermittent basis by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and

analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

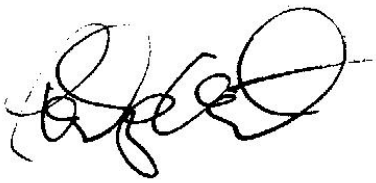
LIMITATIONS

This report has been prepared to aid in the design and construction of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant geotechnical aspects of the project relevant to the design and construction of wastewater plant expansion. In the event that any changes in the design and location of the system as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between explorations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Please contact the undersigned if you have any questions regarding this report.

Submitted for GRI,



H. Stanley Kelsay, PE
Principal

Philip L. Wurst, PE, GE
Senior Engineer

Exp. 6/07

Reference

Priest, G.R., December 1995, Tsunami Hazard Map of the Tillamook Quadrangle, Tillamook, Oregon: Department of Geology and Mineral Industries, Open File Report O-95-67.

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

The subsurface materials and conditions at the site were investigated on September 28 and 29, 2005, with six borings, designated B-1 through B-6. The borings were drilled to depths of between 20.5 and 35.8 ft. The approximate locations of the borings are shown on the Site Plan, Figure 2. An engineer provided by our firm observed and logged the explorations. A detailed description of this work is provided below.

The borings were made with a truck-mounted, Mobile B-52, drill rig provided and operated by Subsurface Technologies of Banks, Oregon, using mud-rotary techniques. Disturbed split-spoon samples and relatively undisturbed Shelby-tube samples were obtained from the borings at 2.5- to 5-ft intervals of depth. The Standard Penetration Test was conducted while taking split-spoon samples. This test consists of driving the sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the Standard Penetration Resistance, or N-value. The N-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency or stiffness of cohesive soils, such as silt. The soil samples obtained in the split-spoon sampler were carefully examined in the field, and representative portions were saved in airtight jars. The ends of Shelby Tube samples were sealed with plastic caps and tape.

An observation standpipe was installed in borings B-2 and B-3 to permit measurements to be made of the depth to the groundwater table. The standpipes consist of a 1-in.-diameter plastic riser pipe inserted to the bottom of the borehole. The lower portion of the standpipe is slotted. The area surrounding the slotted section was backfilled with sand, and the remainder of the boring was sealed with bentonite. Groundwater enters the pipe through the slotted section and rises to a static level. The groundwater level is determined with an electrical probe lowered inside the pipe. The groundwater observation standpipe will need to be decommissioned in accordance with the regulations of the Oregon Water Resources Department before construction of water treatment plant expansion.

Logs of the borings made for this investigation are provided on Figures 1A though 6A; the log of boring B-4 from our geotechnical investigation for the original expansion project in September 2004, is provided on Figure 7A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth where the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N-values are shown graphically, along with the natural moisture contents. The terms used to describe the soil encountered are defined in Table 1A.

LABORATORY TESTING

General

All samples obtained from the borings were returned to our laboratory, where the physical characteristics of the samples were noted, and the field classifications were modified where necessary. Additional laboratory tests included determinations of natural moisture content.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are provided on the Boring Logs, Figures 1A through 7A.

Torvane Shear Strength

The approximate undrained shear strength of relatively undisturbed samples of silt obtained from boring B-5 was determined using the Torvane shear device. The Torvane is a hand-held apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear strength testing are shown on Figure 5A.

Undisturbed Unit Weight

The unit weight of a representative sample of silt (sample S-1, boring B-5) was determined in the laboratory in accordance with ASTM D 2937 by cutting a cylindrical specimen from a Shelby tube sample. The dimensions of the specimen were measured to determine the volume, and the specimen was weighed. After oven-drying, the specimen was weighed again, and the water content was calculated. The dry unit weight was then computed. The test indicates the silt has a dry unit weight of 53 pcf and natural moisture content of 78%.

One-Dimensional Consolidation Test

A one-dimensional consolidation test was performed in conformance with ASTM D 2435 on a relatively undisturbed sample obtained from boring B-5. The test provides data on the compressibility of the underlying fine-grained soils, necessary for settlement studies. The test results are summarized on Figure 8A in the form of a curve showing percent strain versus applied effective stress. The initial and final dry unit weight and moisture content of the sample are also shown on the figure.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane Undrained Shear Strength, tsf
very soft	2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials that exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Boulders
12 - 36 in.

Cobbles
3 - 12 in.

Gravel
1/4 - 3/4 in. (fine)
3/4 - 3 in. (coarse)

Sand
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve

Modifier for Subclassification

Adjective	Percentage of Other Material In Total Sample
clean	0 - 2
trace	2 - 10
some	10 - 30
Sandy, silty, Clayey, etc.	30 - 50

GRI

(medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay - pass No. 200 sieve

Table 2A

GUIDELINES FOR CLASSIFICATION OF ROCK

Relation of RQD and Rock Quality

RQD (Rock Quality Designation), %	(Description of Rock Quality)
0-25	Very poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

Descriptive Terminology for Joint Spacing

Spacing of Joints	Descriptive Term
< 2 in.	Very Close
2 in. - 1 ft	Close
1 ft - 3 ft	Moderately Close
3 ft - 10 ft	Wide
> 10 ft	Very Wide

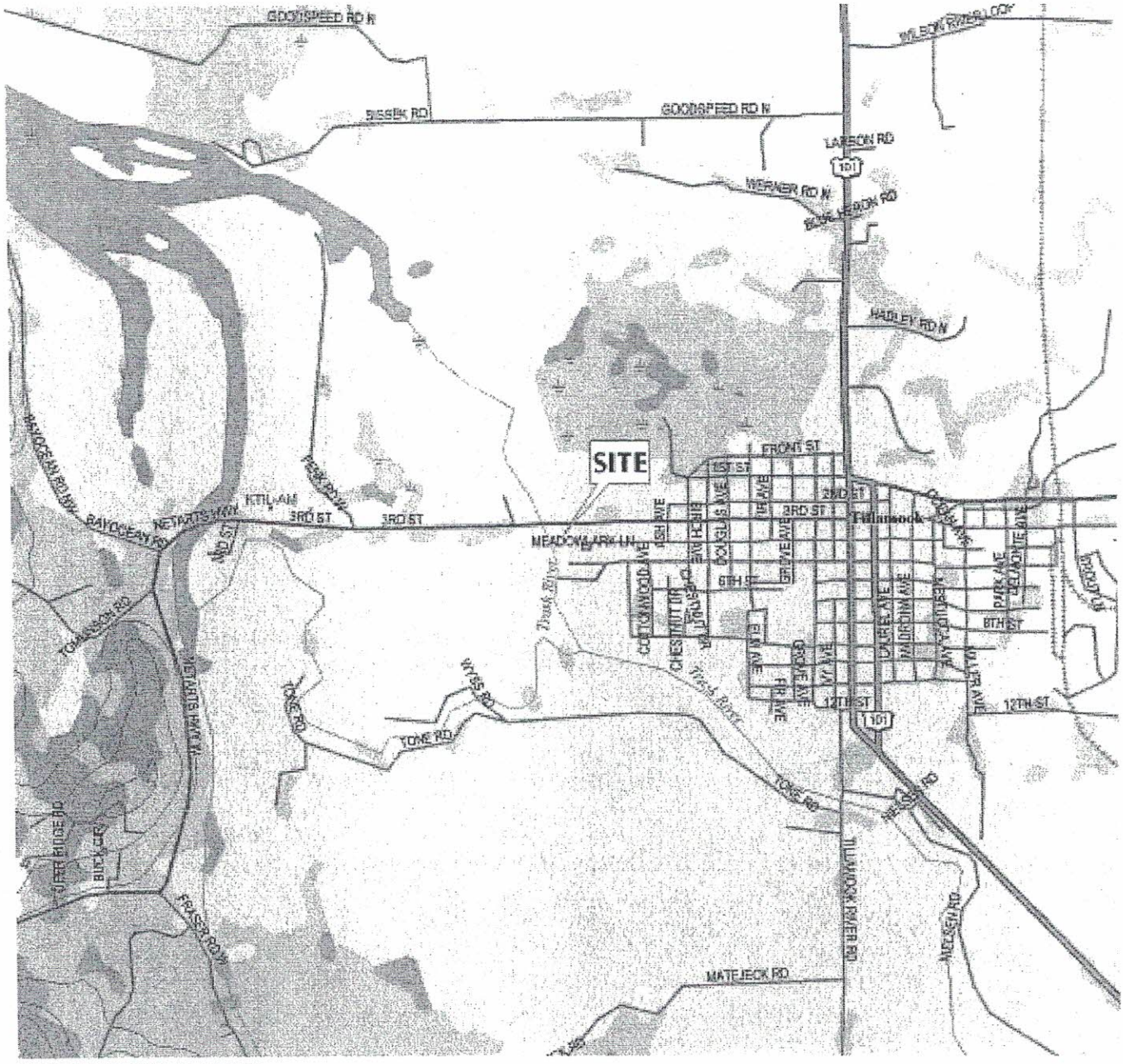
Scale of Rock Hardness (After Panama Canal Company, 1959)

RH-1	Soft	Slightly harder than hard overburden soil, rock-like structure, but crumbles or breaks easily by hand
RH-1	Medium Soft	Cannot be crumbled between fingers, but can be easily picked with light blows of the geology hammer.
RH-2	Medium Hard	Can be picked with moderate blows of geology hammer. Can be cut with knife.
RH-3	Hard	Cannot be picked with geology hammer, but can be chipped with moderate blows of the hammer.
RH-4	Very Hard	Chips can be broken off only with heavy blows of the geology hammer.

Terms Used to Describe the Degree of Weathering

Descriptive Term	Defining Characteristics
Fresh	Rock is unstained. May be fractured, but discontinuities are not stained.
Slight	Rock is unstained. Discontinuities show some staining on their surfaces, but discoloration does not penetrate rock mass.
Moderate	Discontinuity surfaces are stained. Discoloration may extend into rock along discontinuity surfaces.
High	Individual rock fragments are thoroughly stained and can be crushed with pressure hammer. Discontinuity surfaces are thoroughly stained and may be crumbly.

Severe	Rock appears to consist of gravel-sized fragments in a “soil” matrix. Individual fragments are thoroughly discolored and can be broken with fingers.



DELORME 3-D TOPOQUADS, OREGON
TILLAMOOK, OREG. (2ca) 2004

North

0 1/2 1 MILE

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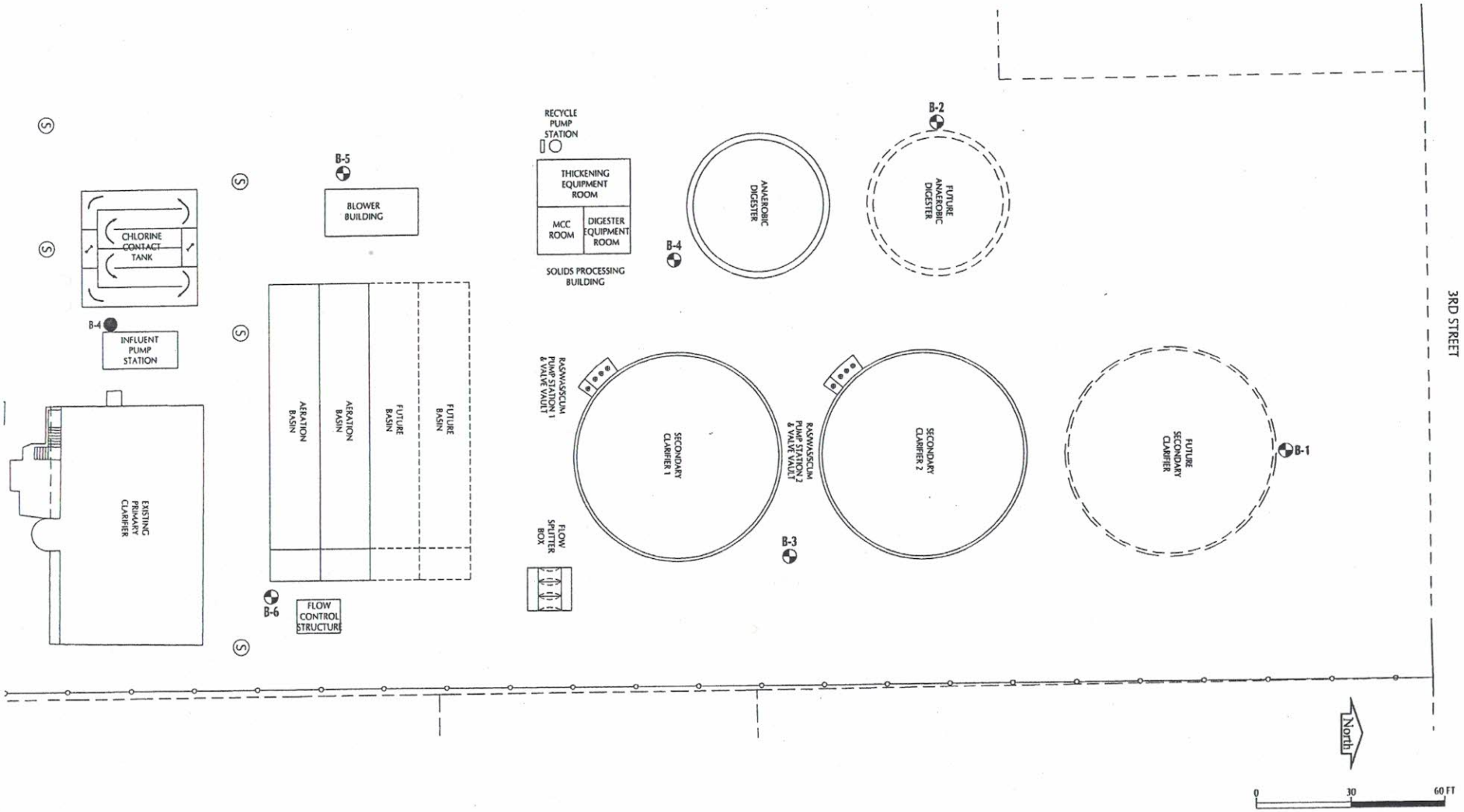
TILLAMOOK WWTP EXPANSION

VICINITY MAP

NOV. 2005 JOB NO. 4144 FIG. 1



BORING MADE BY GRI (SEPTEMBER 28 - 29, 2005)

BORING MADE BY GRI (SEPTEMBER 1, 2004)

SITE PLAN FROM FILE BY KENNEDY / JENKS CONSULTANTS (UNDATED)

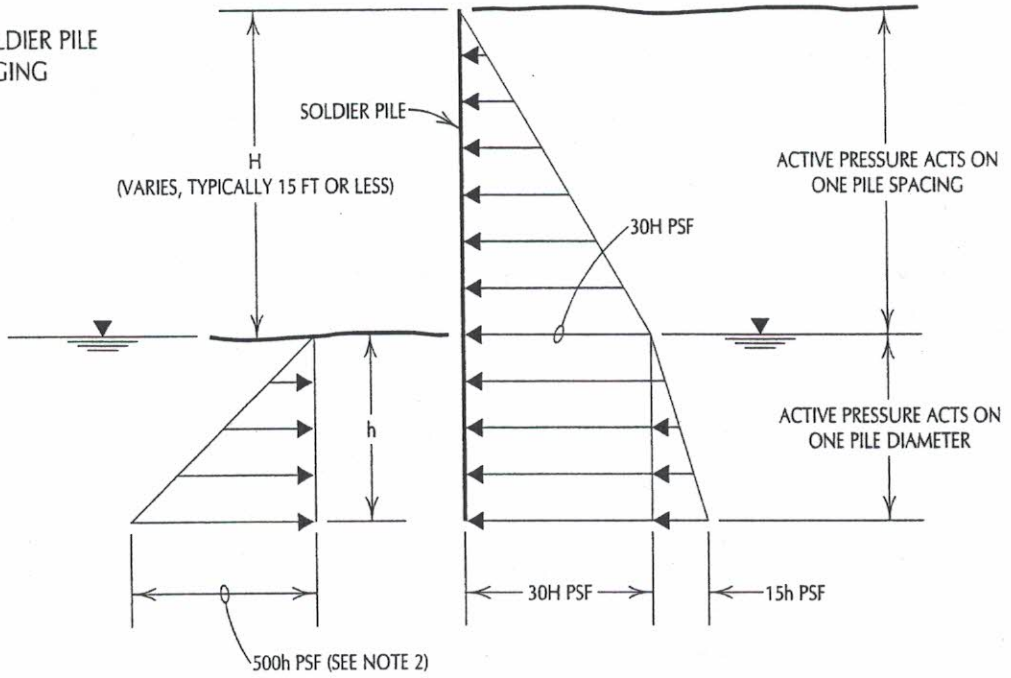
GRI KENNEDY / JENKS CONSULTANTS
TILLAMOOK WWTP EXPANSION

SITE PLAN

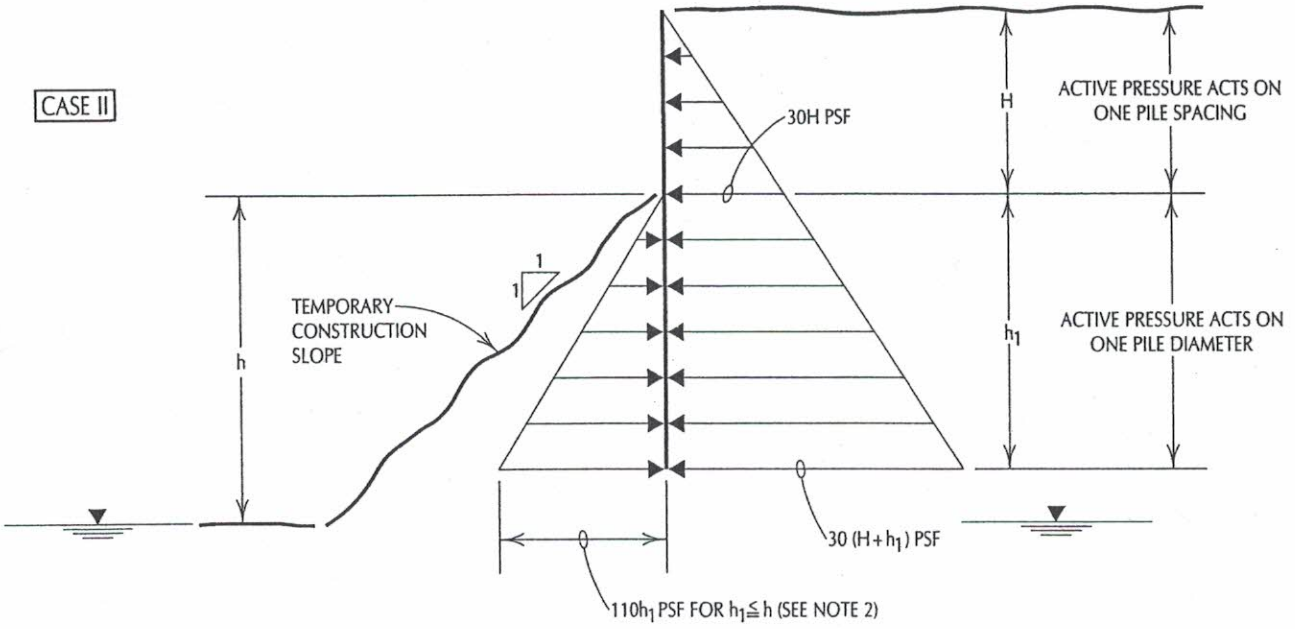
NOV. 2005 JOB NO. 4144 FIG. 2

CANTILEVER SOLDIER PILE
WITH LAGGING

CASE I



CASE II



NOTES:

1) SURCHARGE EFFECTS FROM TRAFFIC, CONSTRUCTION EQUIPMENT, ETC., SHOULD BE ADDED TO THE ABOVE DESIGN PRESSURES. THE ACTUAL AMOUNT OF THIS SURCHARGE; WILL DEPEND ON THE CONTRACTOR'S APPROACH TO THE WORK; HOWEVER, WE RECOMMEND A MINIMUM UNIFORM PRESSURE OF 200 PSF BE USED.

2) PASSIVE RESISTANCE (PSF) ACTING OVER DIAMETER (ACTUAL AREA) OF SOLDIER PILE.

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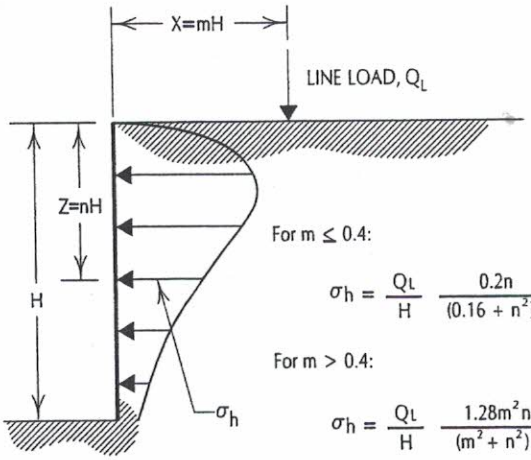
TILLAMOOK WWTP EXPANSION

LATERAL EARTH PRESSURES BRACED SHORING

NOV. 2005 JOB NO. 4144 FIG. 3



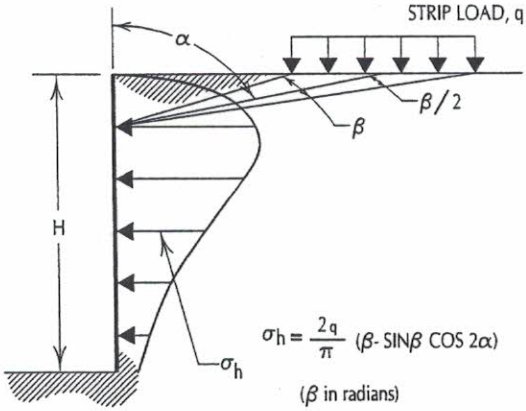
For $m \leq 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{0.2n}{(0.16 + n^2)^2}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_L}{H} \frac{1.28m^2 n}{(m^2 + n^2)^2}$$

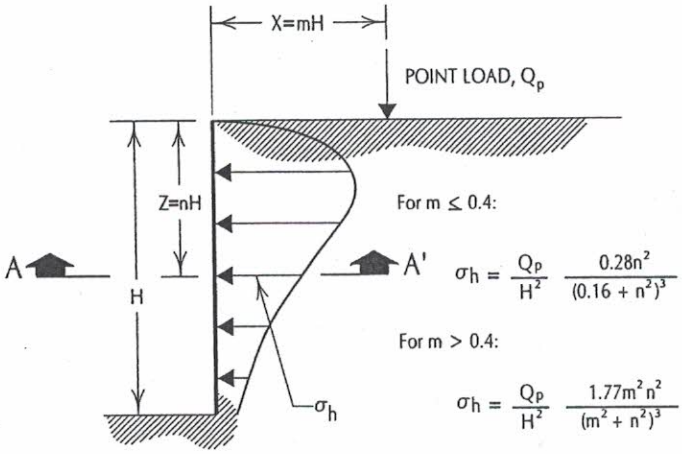
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} (\beta - \sin\beta \cos 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL

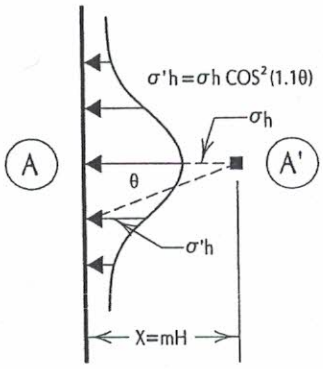


For $m \leq 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28n^2}{(0.16 + n^2)^3}$$

For $m > 0.4$:

$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77m^2 n^2}{(m^2 + n^2)^3}$$



DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.

2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

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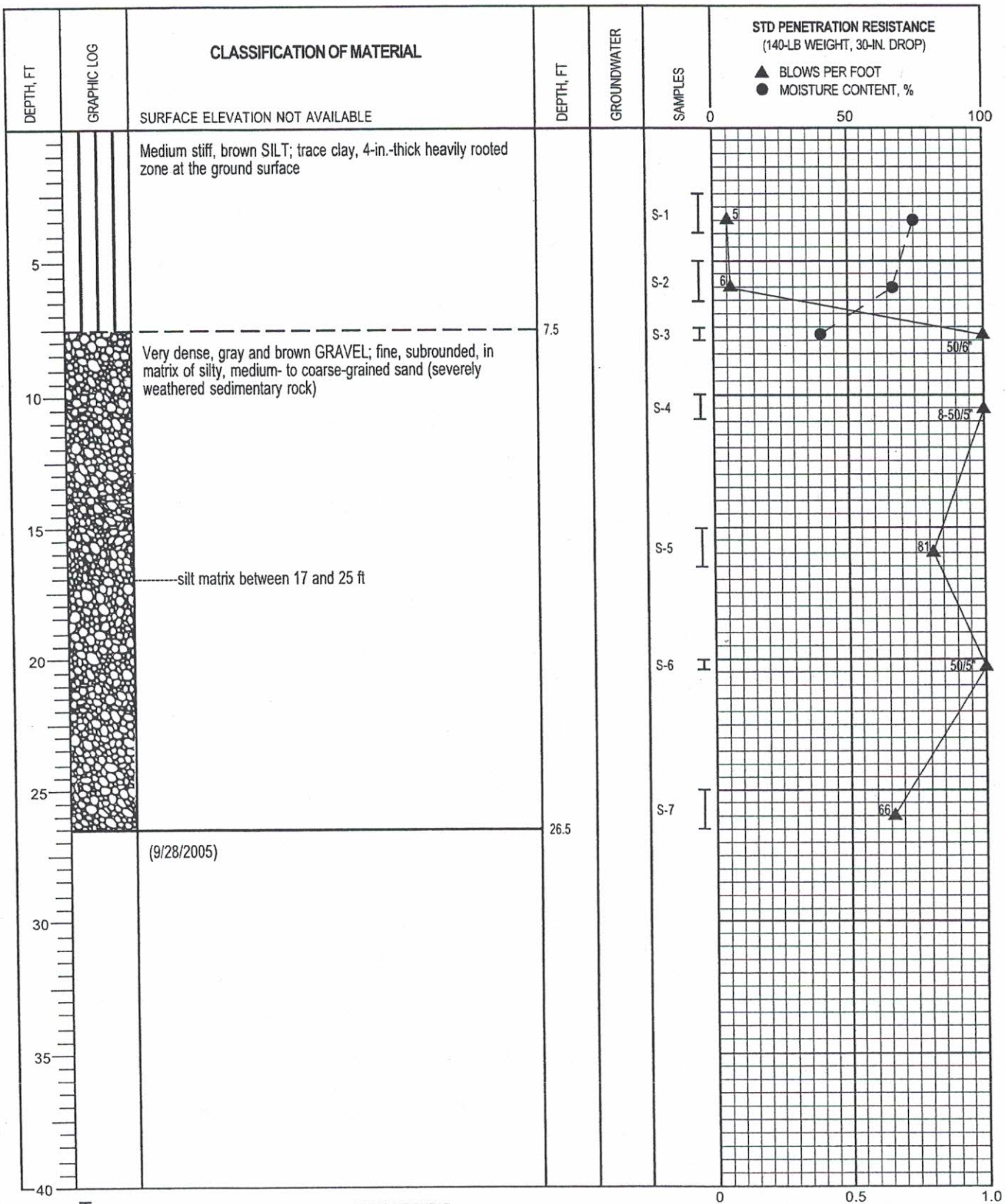
TILLAMOOK WWTP EXPANSION

SURCHARGE-INDUCED LATERAL PRESSURE

NOV. 2005 JOB NO. 4144 FIG. 4



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

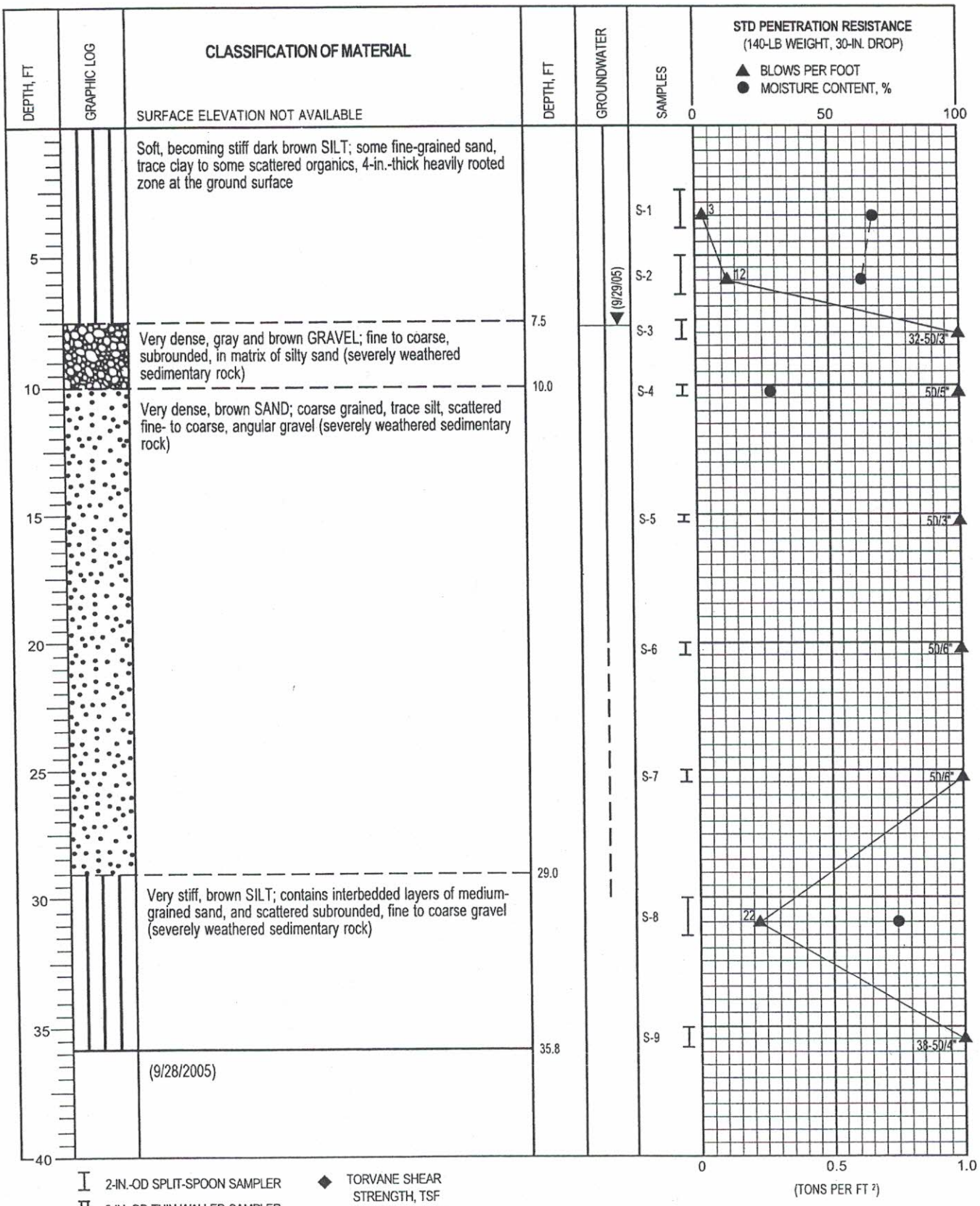
Plastic Limit

GRI

BORING B-1

NOV. 2005 JOB. NO. 4144 FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION NOT AVAILABLE	DEPTH, FT	GROUNDWATER	SAMPLES
		Soft, becoming stiff dark brown SILT; some fine-grained sand, trace clay to some scattered organics, 4-in.-thick heavily rooted zone at the ground surface		(9/29/05)	S-1, S-2, S-3
		Very dense, gray and brown GRAVEL; fine to coarse, subrounded, in matrix of silty sand (severely weathered sedimentary rock)	7.5		S-3
		Very dense, brown SAND; coarse grained, trace silt, scattered fine- to coarse, angular gravel (severely weathered sedimentary rock)	10.0		S-4, S-5, S-6, S-7
		Very stiff, brown SILT; contains interbedded layers of medium-grained sand, and scattered subrounded, fine to coarse gravel (severely weathered sedimentary rock)	29.0		S-8, S-9
		(9/28/2005)	35.8		



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

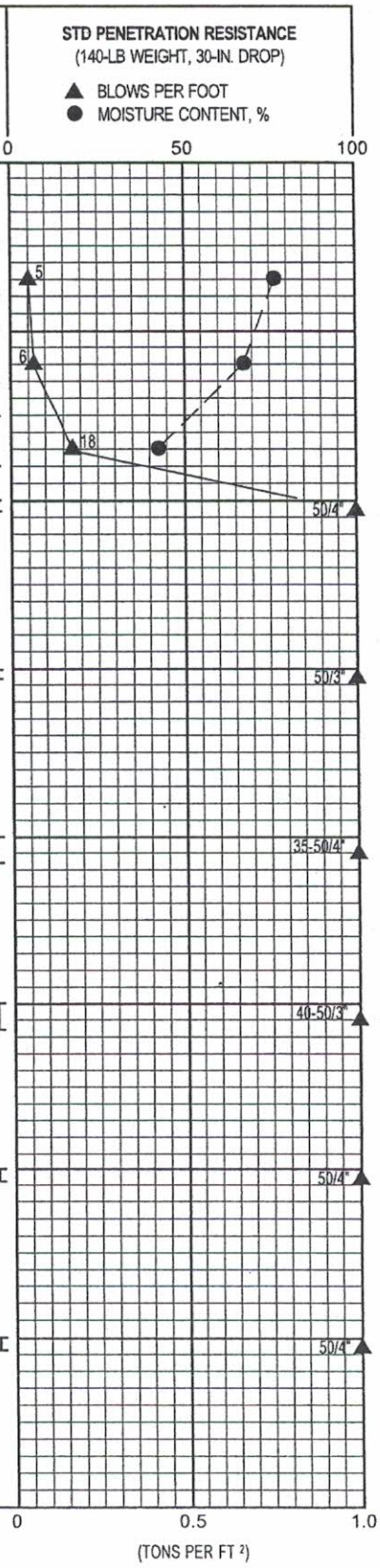
Plastic Limit

GRI

BORING B-2

NOV. 2005 JOB. NO. 4144 FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION NOT AVAILABLE	DEPTH, FT	GROUNDWATER	SAMPLES
		Medium stiff, light to dark brown SILT; trace to some clay, scattered organics, 4-in.-thick heavily rooted zone at the ground surface			S-1, S-2, S-3
		Medium dense, brown SAND; medium grained, trace silt, scattered subrounded to rounded, fine gravel (severely weathered sedimentary rock)	7.5		S-4
		Very dense, gray and brown GRAVEL; fine, subrounded, in matrix of medium- to coarse-grained sand (severely weathered sedimentary rock)	12.5		S-5, S-6, S-7, S-8, S-9
		(9/29/2005)	35.4		



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

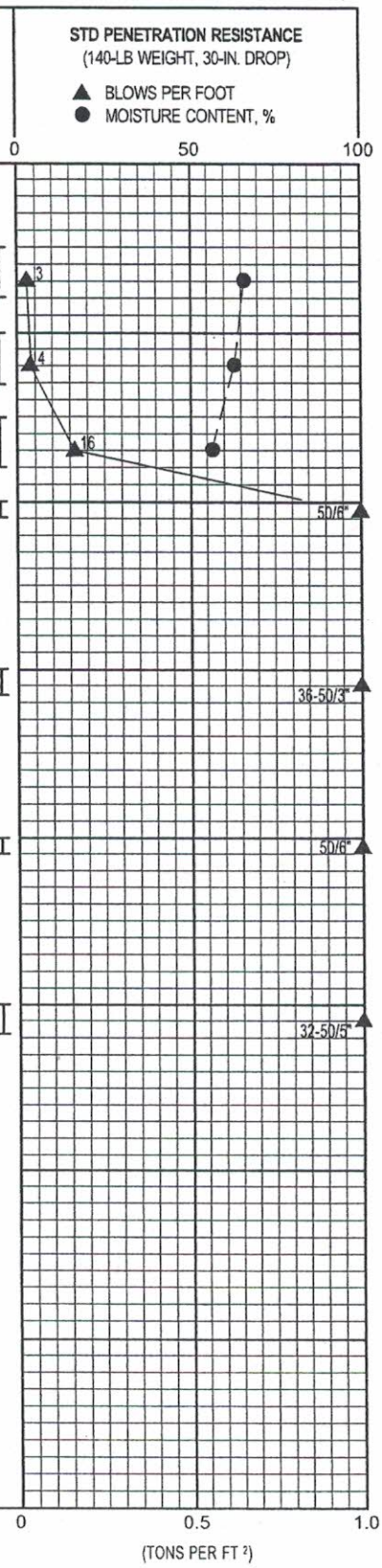
Plastic Limit

GRI

BORING B-3

NOV. 2005 JOB. NO. 4144 FIG. 3A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION NOT AVAILABLE	DEPTH, FT	GROUNDWATER	SAMPLES
		Soft to medium stiff, brown SILT; trace fine-grained sand, 4-in.-thick heavily rooted zone at the ground surface			S-1, S-2
		Medium dense, brown, silty SAND; fine grained, scattered rounded, fine gravel	7.5 8.5		S-3
		Very dense, gray and brown GRAVEL; fine, subrounded, in matrix of coarse-grained sand (severely weathered sedimentary rock)			S-4, S-5, S-6, S-7
		(9/28/2005)	26.0		



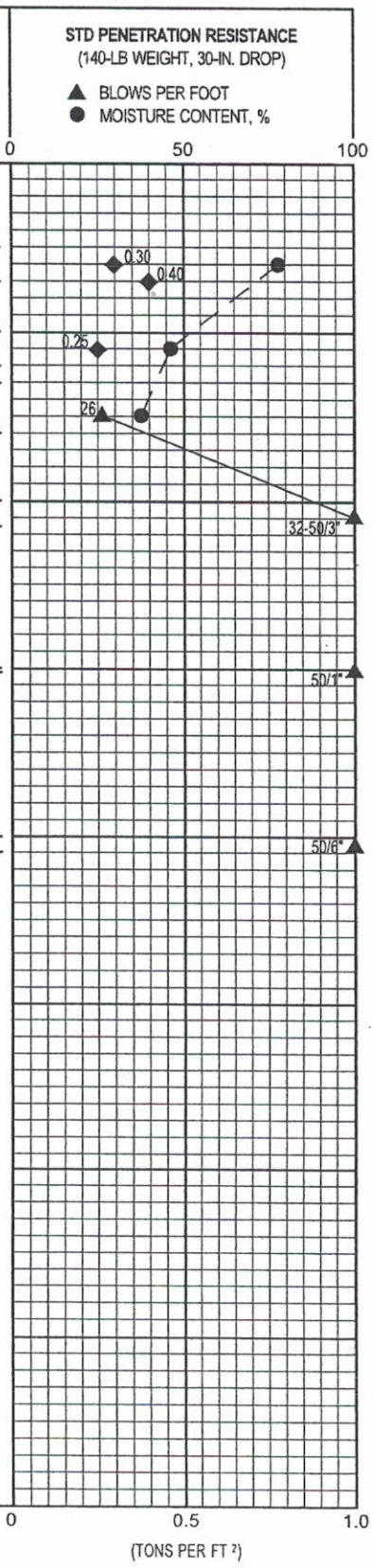
2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)
TORVANE SHEAR STRENGTH, TSF
UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-4

NOV. 2005 JOB. NO. 4144 FIG. 4A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION NOT AVAILABLE	DEPTH, FT	GROUNDWATER	SAMPLES
		Medium stiff, brown SILT; trace clay and fine-grained sand, 4-in.-thick heavily rooted zone at the ground surface			S-1, S-2, S-3
		Medium to very dense, brown mottled orange SAND; medium to coarse grained, some silt, scattered fine, subrounded to rounded gravel (severely weathered sedimentary rock)	6.5		S-4, S-5 *, S-6
		(9/29/2005)	20.5		



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

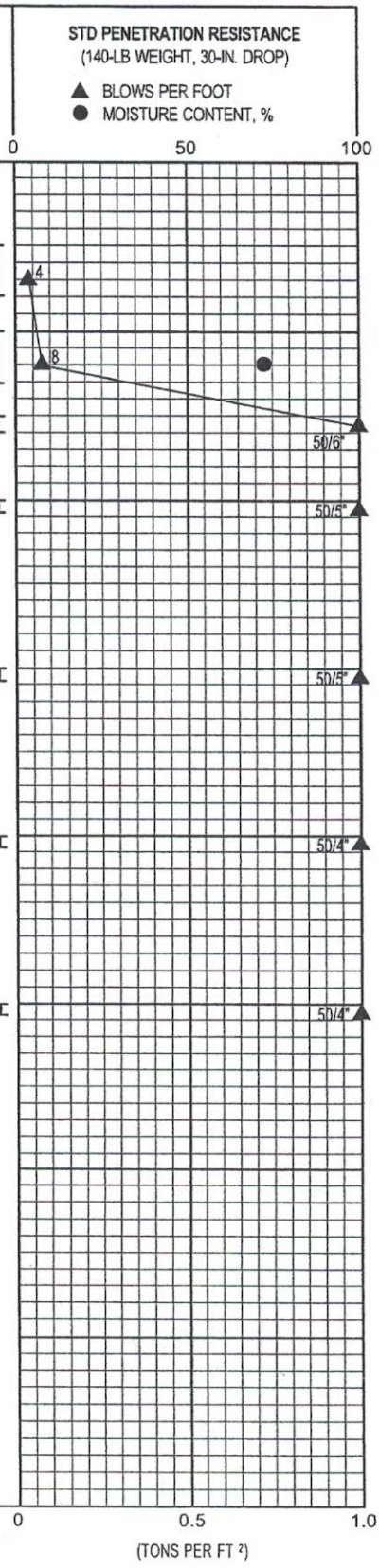
Plastic Limit

GRI

BORING B-5

NOV. 2005 JOB. NO. 4144 FIG. 5A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION NOT AVAILABLE	DEPTH, FT	GROUNDWATER	SAMPLES
		Medium stiff to stiff, brown SILT; some fine-grained sand, trace clay, 4-in.-thick heavily rooted zone at the ground surface			S-1 *
5					S-2
			7.5		S-3
10		Very dense, brown mottled orange SAND; medium to coarse, scattered fine, subrounded to rounded gravel (severely weathered sedimentary rock)			S-4
15					S-5
20					S-6
25			25.3		S-7
		(9/29/2005)			
30					
35					
40					



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

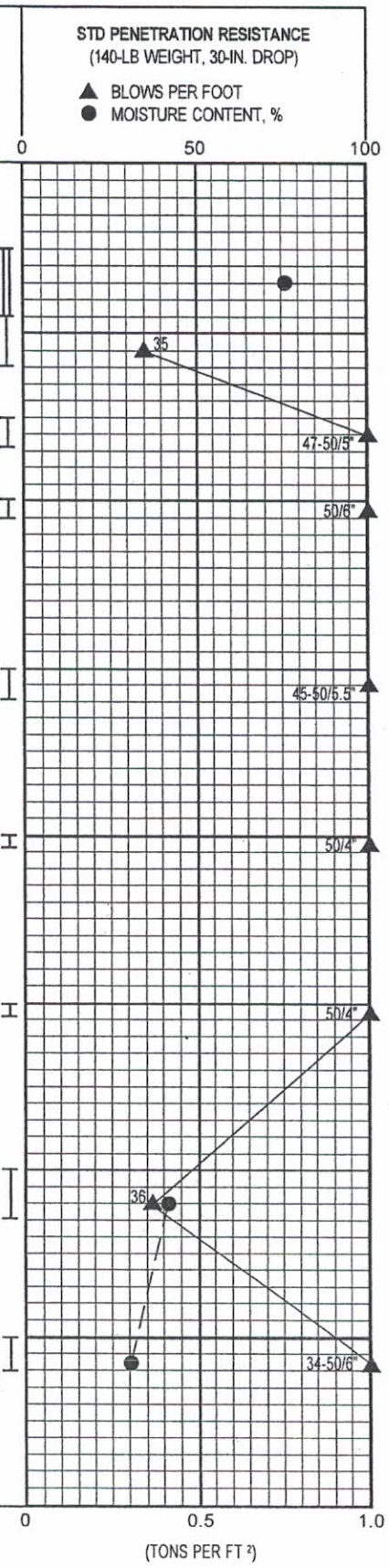
Plastic Limit

GRI

BORING B-6

NOV. 2005 JOB. NO. 4144 FIG. 6A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION NOT AVAILABLE	DEPTH, FT	GROUNDWATER	SAMPLES
		Dense, gravelly SAND; fine to coarse grained, some silt, 4-in.-thick heavily rooted zone at the ground surface (severely weathered sedimentary rock)			S-1, S-2, S-3, S-4, S-5, S-6, S-7, S-8, S-9
30		Hard, reddish-brown SILT; some sand, some fine to coarse, rounded gravel (severely weathered sedimentary rock)	30.0		
35		(9/1/2004)	36.0		



2-IN.-OD SPLIT-SPOON SAMPLER

3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

TORVANE SHEAR STRENGTH, TSF

UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

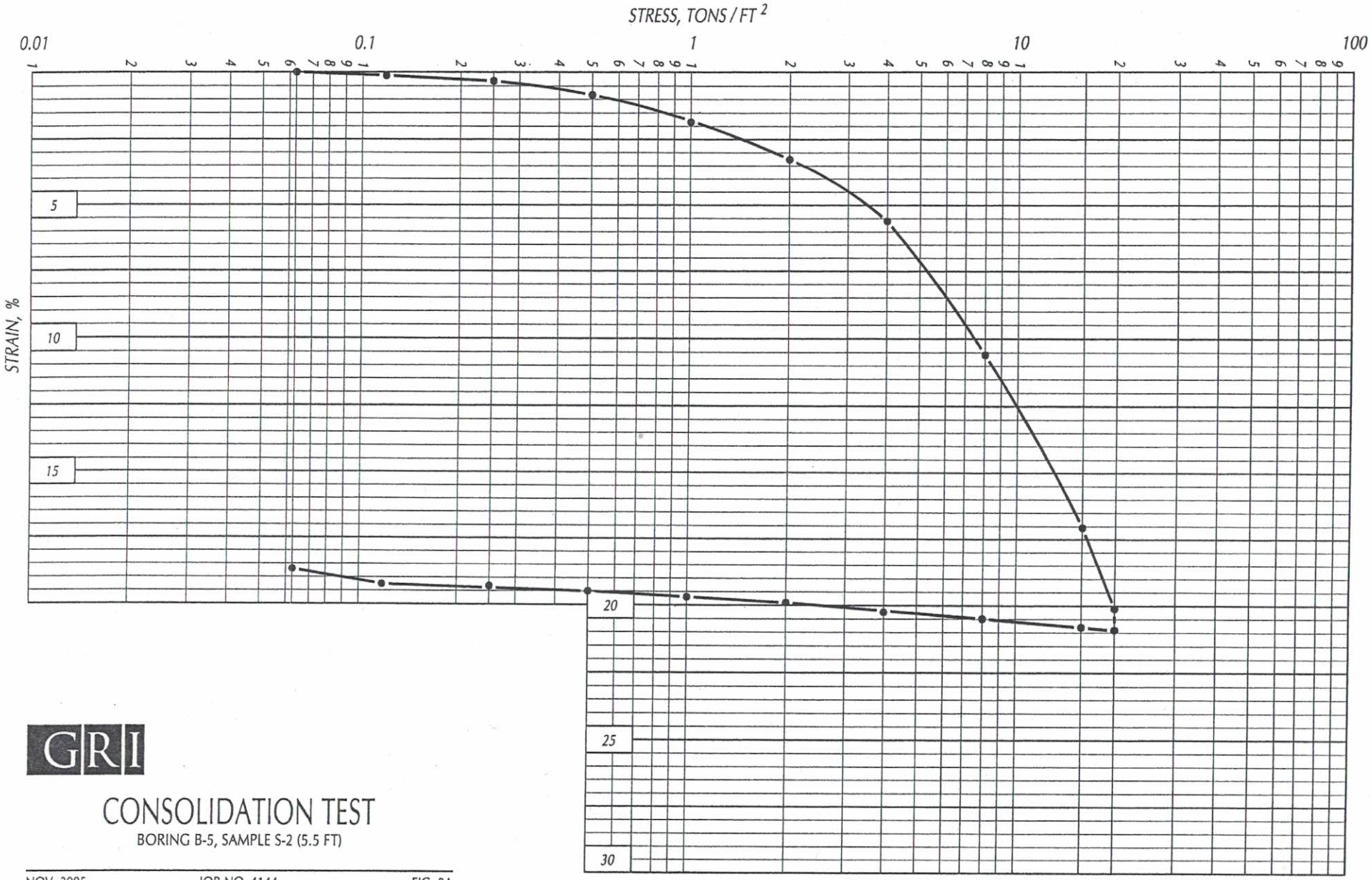
GRI

BORING B-4

(1994)

NOV. 2005 JOB. NO. 4144 FIG. 7A

BORING	SAMPLE	DEPTH, FT	MOISTURE CONTENT, % (INITIAL)	MOISTURE CONTENT, % (FINAL)	DRY UNIT WEIGHT, PCF (INITIAL)	DRY UNIT WEIGHT, PCF (FINAL)	SOIL DESCRIPTION
B-5	S-2	5.5	62	46	60	73	MEDIUM STIFF, BROWN SILT; TRACE CLAY AND FINE-GRAINED SAND



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CONSOLIDATION TEST

BORING B-5, SAMPLE S-2 (5.5 FT)

NOV. 2005 JOB NO. 4144 FIG. 8A