

## Part 3 Project Feasibility

### 5.0 Basis of Planning

#### 5.1 Regulatory Requirements

##### 5.1.1 Permit Constraints

This section summarizes the NPDES waste discharge requirements for the MCSD WWMF. MCSD currently discharges under NPDES Permit No. CA0024490, Waste Discharge Order No. WQ 2011-0008-DWQ. This permit was adopted by the State Water Resources Control Board (SWRCB) on April 19, 2011, and went into effect on the same day. The permit will expire on April 18, 2016.

##### 5.1.1.1 Discharge Prohibitions

Pursuant to SWRCB Order No. WQ 2011-0008-DWQ, the discharge of treated effluent from the WWMF to the Mad River or its tributaries is prohibited from May 15 through September 30 of each year. This discharge prohibition does not prohibit discharge to the Hiller Storm Water Treatment Wetlands (Discharge Point 005) or the percolation ponds (Discharge Point 002). From October 1 through May 14 of any given year, treated wastewater effluent from the WWMF can be discharged directly to the Mad River when river flows are both greater than 200 cfs and greater than 100 times the wastewater discharge rate, based on the most recent daily flow measurement, as measured at the Highway 299 overpass (USGS Gage No. 11-4810.00).

##### 5.1.1.2 Effluent Limitations

With the adoption of the 2011 NPDES permit, monitoring locations were revised for the facility discharges. Table 5-1 summarizes the monitoring locations for compliance with the effluent limitations. Discharge points are shown on Figure 5-1. Monitoring locations, including surface water, land reclamation, receiving water and monitoring well locations, are shown on Figure 5-2.

<b>Table 5-1</b> <b>Discharge Monitoring Locations<sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>		
<b>Discharge Point</b>	<b>Monitoring Location</b>	<b>Monitoring Location Description</b>
--	M-INF	Treatment facility headworks
All	M-001	Chlorine contact chamber following dechlorination
001	M-002	Outfall to the Mad River under the Hammond Trail railroad bridge
002	M-003	Outfall to Mad River percolation ponds
003	M-004	Recycled wastewater irrigation of Lower Fisher Ranch
004	M-005	Discharge to land on Upper Fisher Ranch
005	M-006	Recycled wastewater irrigation of Hiller Storm Water Treatment Wetland
006	M-007	Recycled wastewater irrigation of Pialorsi Ranch
--	M-008	Overflow from the Hiller Storm Water Treatment Wetland
--	R-001	Mad River at Highway 101 Bridge



\\Eureka\Projects\2008\008189-MCSD\300-FacilitiesPlan\Drawings\SAVED: 8/5/2011 2:22 PM CNEWELL, PLOTTED: 8/8/2011 8:45 AM, CHRIS D. NEWELL



SOURCE: 2010 NAIP IMAGERY

**SHN**  
Consulting Engineers  
& Geologists, Inc.

McKinleyville Community Services District  
Wastewater Management Facility  
McKinleyville, California

August 2011

008189-DISCHRG-LOC

Dishcharge Location Map

SHN 008189

Figure 5-1



\\Eureka\Projects\2008\008189-MCSD\300-FacilitiesPlan\Drawings\SAVED: 8/5/2011 2:23 PM CNEWELL, PLOTTED: 8/8/2011 8:46 AM, CHRIS D. NEWELL



SOURCE: 2010 NAIP IMAGERY

**SHN**  
Consulting Engineers  
& Geologists, Inc.

McKinleyville Community Services District  
Wastewater Management Facility  
McKinleyville, California

August 2011

008189-MWELL-LOC

Monitoring Location Map

SHN 008189

Figure 5-2



<b>Table 5-1</b> <b>Discharge Monitoring Locations<sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>		
<b>Discharge Point</b>	<b>Monitoring Location</b>	<b>Monitoring Location Description</b>
--	R-002	North bank of Mad River as close as possible to the discharge point under the Hammond Trail Bridge
--	W-001	Well M-1 adjacent to Fisher Road
--	W-002	Well M-2 on the SW corner of the intersection of School and Fisher Roads
--	W-006	Well M-6 south of W-9 and west of W-7
--	W-007	Well M-7 in the upper portion of the Fisher parcel
--	W-008	Well M-8 400 feet west of the intersection of School and Fisher Roads
--	W-009	Well M-9 adjacent to School Road
--	W-014	Well downgradient of the Hiller Storm Water Treatment Wetland irrigation area
--	W-015	Well within the Lower Fisher Ranch irrigation area
--	W-016	Well within the Pialorsi Ranch irrigation area
1. Reproduced from NPDES No. CA0024490, Attachment E: Monitoring and Reporting Program		

Table 5-2 summarizes the effluent limitations for discharges from the WWMF to the Mad River.

<b>Table 5-2</b> <b>Wastewater Effluent Limitations for Discharge Point 001(Mad River Outfall)<sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>								
<b>Parameter</b>	<b>Units</b>	<b>Monthly Average<sup>2</sup></b>	<b>Weekly Average<sup>3</sup></b>	<b>Daily Max.</b>	<b>Instantaneous</b>		<b>Sampling</b>	
					<b>Min.</b>	<b>Max.</b>	<b>Type</b>	<b>Frequency</b>
BOD <sup>4</sup>	mg/L <sup>5</sup>	45	65	--	--	--	24-hr	Weekly
	ppd <sup>6,7</sup>	604	873	--	--	--	Composite	
TSS <sup>8</sup>	mg/L	83	--	--	--	--	24-hr	Weekly
	ppd <sup>7</sup>	1,108	--	--	--	--	Composite	
pH	unitless	--	--	--	6.5	8.5	Grab	Daily
Settleable Matter	ml/L <sup>8</sup>	0.1	--	0.2	--	--	Grab	Weekly
Chlorine Residual	mg/L	0.01	--	0.02	--	--	Grab	Daily
Nitrate as Nitrogen	mg/L	10	--	--	--	--	Grab	Monthly
4,4'-DDT <sup>9</sup>	µg/L <sup>10</sup>	0.00059	--	0.0027	--	--	Grab	Semi-Annually
Bis(2-ethylhexyl) phthalate	µg/L	1.8	--	3.0	--	--	Grab	Semi-Annually
Total Coliform	MPN/100 ml <sup>11</sup>	23 (median)	--	230	--	--	Grab	Weekly
1. Reproduced from NPDES No. CA0024490 2. The arithmetic mean of all daily results during a calendar month 3. The arithmetic mean of all daily results made during a calendar week 4. BOD: Biochemical Oxygen Demand 5. mg/L: milligrams per liter 6. ppd: pounds per day 7. Based on a design flow rate of 1.61 Million Gallons per Day (MGD). 8. ml/L: milliliters per Liter 9. 4,4'-DDT: Dichlorodiphenyltrichloroethane 10. µg/L: micrograms per Liter 11. MPN/100 ml: Most Probable Number per 100 milliliters								

In addition to the effluent limitations listed in Table 5-2, the permit requires that the average monthly percent removal of BOD and TSS shall not be less than 65% as measured at Monitoring Location M-001. The percent removal shall be determined from the monthly average influent concentrations and monthly average effluent concentrations for each constituent over the same period.

The permit also sets forth acute toxicity effluent limitations when discharging to the Mad River. The limitations state that no acute toxicity shall be present in the effluent. Discharges are considered in compliance with this requirement when the survival of aquatic organisms in a 96-hour bioassay meets a minimum 70% survival for any one bioassay and a median of at least 90% survival for all bioassays conducted in a calendar month.

Effluent limitations for discharges to the percolation ponds are shown in Table 5-3 and the effluent limitations for the land reclamation sites are shown in Table 5-4.

### 5.1.1.3 Receiving Water Limitations

Receiving waters are monitored in the Mad River at the Highway 101 bridge upstream of the influence of the discharge (R-001) and on the north bank of the Mad River as close as possible to the discharge point under the Hammond Trail Bridge (R-002). Receiving water samples collected at these locations are compared to receiving water limitations based on the water quality objectives contained in the basin plan for the Mad River. The receiving water limitations address water quality objectives for dissolved oxygen, specific conductance, pH, turbidity, floatables, taste- and odor-producing substances, coloration, bottom deposits (total dissolved solids), biostimulants, toxic substances, temperature, pesticides, oils/grease, and other chemical constituents as specified in the basin plan.

<b>Table 5-3</b> <b>Wastewater Effluent Limitations for Discharge Point 002 (Percolation Ponds)<sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>								
Parameter	Units	Monthly Average <sup>2</sup>	Weekly Average <sup>3</sup>	Daily Max.	Instantaneous		Sampling	
					Min.	Max.	Type	Frequency
BOD <sup>4</sup>	mg/L <sup>5</sup>	45	65	--	--	--	24-hr Composite	Weekly
TSS <sup>6</sup>	mg/L	83	--	--	--	--	24-hr Composite	Weekly
Nitrate as Nitrogen	mg/L	10	--	--	--	--	Grab	Monthly
Total Coliform	MPN/100 ml <sup>7</sup>	23 (median)	--	230	--	--	Grab	Weekly
1. Reproduced from NPDES No. CA0024490 2. The arithmetic mean of all daily results during a calendar month 3. The arithmetic mean of all daily results made during a calendar week					4. BOD: Biochemical Oxygen Demand 5. mg/L: milligrams per liter 6. TSS: Total Suspended Solids 7. MPN/100 ml: Most Probable Number per 100 milliliters			

<b>Table 5-4</b> <b>Wastewater Effluent Limitations for Discharge Points 003, 004, 005, and 006 (Land Reclamation) <sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>								
Parameter	Units	Monthly Average <sup>2</sup>	Weekly Average <sup>3</sup>	Daily Max.	Instantaneous		Sampling	
					Min.	Max.	Type	Frequency
BOD <sup>4</sup>	mg/L <sup>5</sup>	45	65	--	--	--	24-hr Composite	Weekly
TSS <sup>6</sup>	mg/L	83	--	--	--	--	24-hr Composite	Weekly
Total Coliform	MPN/100 ml <sup>7</sup>	23 (median)	--	230	--	--	Grab	Weekly
1. Reproduced from NPDES No. CA0024490 2. The arithmetic mean of all daily results during a calendar month 3. The arithmetic mean of all daily results made during a calendar week 4. BOD: Biochemical Oxygen Demand 5. mg/L: milligrams per liter 6. TSS: Total Suspended Solids 7. MPN/100 ml: Most Probable Number per 100 milliliters								

#### 5.1.1.4 New Provisions

Order No. WQ 2011-0008-DWQ rescinded the previous NPDES permit (Order No. R1-2008-0039) and contains the following significant changes:

1. Effluent limitations for copper, lead, alpha-BHC and dioxin congeners were removed from the permit.
2. Effluent limitations for 4,4' DDT have been revised.
3. Mass-based limits for the BOD and TSS have been revised.
4. Receiving water classifications have been removed from the permit for discharges to the Hiller Stormwater Wetlands and the Lower Fisher Ranch Stormwater Ditch.

## 5.1.2 Pre-treatment Regulations

### 5.1.2.1 Source Control Program

The District is required to perform source control functions, including the following:

- Implement the necessary legal authorities to monitor and enforce source control standards, restrict discharges of toxic materials to the collection system and inspect facilities connected to the system.
- If waste haulers are allowed to discharge to the Facility, establish a waste hauler permit system, to be reviewed by the Executive Officer, to regulate waste haulers discharging to the collection system of Facility.

- Conduct a waste survey once every five years, or more frequently if required by the Regional Water Board Executive Officer, to identify all industrial dischargers that might discharge pollutants that could pass through or interfere with the operation or performance of the Facility.
- Perform ongoing industrial inspections and monitoring, as necessary, to ensure adequate source control.

### **5.1.2.2 Summary of Recent Source Control Studies**

During a pretreatment compliance audit conducted by Tetra Tech in 2009 it was determined the District local limits were not adequate (Tetra Tech, 2009). As part of the process to update the local limits, it was also determined that the District Sewer Use Ordinance would need to be updated to give the District better authority to enforce new local limits and to establish individual discharge permits for commercial customers that have the potential to discharge wastes other than domestic sewerage. A copy of the RWQCB Correspondence and the pretreatment compliance inspection summary report prepared by Tetra Tech in April 2009 is included in Appendix F.

The District enlisted the services of Freshwater Environmental Services (FES) to complete a Sanitary Sewer Monitoring Program Report in June 2009 (FES, 2009). FES was also contracted to draft a sewer use ordinance and to complete a local limits work plan for RWQCB for concurrence. A draft local limits development work plan was prepared by FES in January 2011 (FES, 2011a). Copies of the additional source control studies completed by FES are included in Appendix F.

## **5.1.3 Collection System Standards**

### **5.1.3.1 Statewide General WDRs for Sanitary Sewer Systems**

On May 2, 2006, the State Water Board adopted State Water Board Order 2006-0003-DWQ, Statewide General WDRs for Sanitary Sewer Systems. Order No. 2006-0003-DWQ requires that all public agencies that currently own or operate sanitary sewer systems apply for coverage under the General WDRs. The deadline for dischargers to apply for coverage under State Water Boards Order 2006-0003-DWQ was November 2, 2006. The District has applied for coverage under, and is subject to the requirements of Order 2006-0003-DWQ and any future revisions thereto for operation of its wastewater collection system.

In addition to the coverage obtained under Order 2006-0003, the Discharger's collection system is also part of the treatment system that is subject to this Order. As such, pursuant to federal regulations, the discharger must properly operate and maintain its collection system (40 CFR § 122.41(e)), report any non-compliance (40 CFR § 122.41(l)(6) and (7)), and mitigate any discharge from the collection system in violation of this Order (40 CFR § 122.41(d)).

### **5.1.3.2 Sanitary Sewer Overflows**

The District is required to continue electronic and/or telefax reporting of Sanitary Sewer Overflows (SSOs) pursuant to Provision D.15 and General Monitoring and Reporting

Requirement G.2 of Order No. 2006-0003-DWQ and Monitoring and Reporting Program No. 2006-0003-DWQ. Oral reporting of SSOs as specified in the NPDES permit is required through the term of the Order.

FES recently completed a Sanitary Sewer Management Plan for MCSD in May 2011 (FES, 2011b) in compliance with the general requirements of Order No. 2006-003. A copy of the plan is included in Appendix G.

#### **5.1.4 Reclaimed Water Use Regulations**

The District is required to comply with applicable state and local requirements regarding the production and use of reclaimed wastewater, including requirements of Water Code sections 13500–13577 (Water Reclamation) and Department of Health Services regulations at CCR Title 22, Sections 60301–60357 (Water Recycling Criteria).

The District is also required to maintain compliance with the effluent limitations listed in Table 5-4 when applying effluent at Discharge Points 003, 004, 005, and 006. Compliance is measured at Monitoring Location M-001.

In addition to the effluent limitations listed the following reclamation specifications also apply in the permit:

- Disinfection: The disinfected effluent shall not contain concentrations of total coliform bacteria exceeding the following concentrations:
  - The median concentration shall not exceed 23 MPN/100 ml, for samples collected during any calendar month.
  - No sample shall exceed a coliform count of 230 MPN/100 ml.
- The use of recycled water shall not create a condition of pollution or nuisance as defined in Water Code section 13050(m).
- Recycled water and airborne spray shall not be allowed to escape from the authorized recycled water use area(s). [CCR Title 22, Section 60310(e)]
- Direct or windblown spray, mist, or runoff from irrigation areas shall not enter dwellings, designated outdoor eating areas, or food handling facilities. [CCR title 22, section 60310(e)(2)]
- Disinfected secondary treated recycled water shall not be irrigated within 100 feet of any domestic water supply well or domestic water supply surface intake, unless the technical requirements specified in CCR Title 22, Section 60310(a) have been met and approved by the Department of Health Services (DHS).
- Disinfected secondary treated recycled water shall not be irrigated with 100 feet of the change in grade between the upper and lower Fisher Ranch irrigation areas. Best management practices shall also be developed and implemented to prevent the creation of runoff that leads to the discharge of recycled water to the County drainage swale located on the Lower Fisher Ranch.



- All areas where recycled water is used that are accessible to the public shall be posted with signs that are visible to the public, in a size no less than 4 inches high by 8 inches wide, that include the following wording: "RECYCLED WATER-DO NOT DRINK" (CCR Title 22, Section 60310(g)). Each sign shall display an international symbol similar to that shown in Title 22, Figure 60310-A. These warning signs shall be posted at least every 500 feet with a minimum of a sign at each corner and access road.

## 5.1.5 Biosolids Regulations

### 5.1.5.1 List of Regulations

MCSD's disposal of biosolids is currently regulated under NPDES Permit No. CA0024490 Order No. WQ 2011-0008-DWQ. Order No. 2011-0008-DWQ states that biosolids shall be disposed of in accordance with applicable federal and state regulations.

As set forth in the permit, the disposal of biosolids is regulated through the following requirements:

- 40 CFR Parts 257, 258, 501, and 503; EPA's Biosolids Rule
- SWRCB Order No. 2004-0012-DWQ (General Waste Discharge Requirements for the Discharge of Biosolids to Land as a Soil Amendment in Agricultural, Silvicultural, Horticultural, and Land Reclamation Activities (General Order)
- CCR Title 27, Division 2

Sludge or biosolids that are disposed of in a municipal solid waste landfill or used as landfill cover are required to meet the applicable requirements of 40 CFR 258.

The 40 CFR Part 503 regulations for land application fall into two broad categories: Processes to Reduce Pathogens and Vector Attraction Reduction. The application of Processes to Significantly Reduce Pathogens (PSRP) and Processes to Further Reduce Pathogens (PFRP) involves the deactivation of pathogens and results in two categories of biosolids.

- Class A biosolids are high quality sludges, low in pathogens (less than 1,000 Fecal Coliform (FC)/100 ml), suitable to be sold or given away for a variety of purposes including home gardens and lawns, silviculture, and land not meeting site restrictions for Class B biosolids.
- Class B biosolids are suitable for pasture and woodland applications, non-contact use only (less than 2,000,000 FC/100 ml).

The SWRCB General Order sets forth WDRs for use of Class B biosolids as soil amendment for agricultural, silvicultural, horticultural, and land reclamation applications. The General Order is intended to streamline the regulatory process for application of biosolids; however, it does not supersede 40 CFR Part 503 Regulations. Class A biosolids complying with the 40 CFR Part 503 are not regulated by the General Order except under special circumstances.

### 5.1.5.2 Summary of 40 CFR Part 503 Regulations

All land-applied biosolids must comply with one of the pathogen reduction standards listed in 40 CFR Part 503.32. Table 5-5 summarizes the pathogen reduction standards.

<b>Table 5-5</b> <b>Summary of Pathogen Reduction Requirements<sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>	
<b>Class A Biosolids<sup>2</sup></b>	<b>Class B Biosolids<sup>3</sup></b>
Alternative 1: Thermally Treated Biosolids. Use one of four time-temperature regiments.	Alternative 1: Monitoring of Indicator Organisms. Test for fecal coliform density as an indicator for all pathogens at the time of biosolids use or disposal.
Alternative 2: Biosolids Treated in a High pH-High Temperature Process. Specifies pH, temperature, and air-drying requirements.	Alternative 2: Use of Processes to Significantly Reduce Pathogens (PSRP). Biosolids are treated in one of the PSRP identified in CFR <sup>4</sup> 40 Part 503.
Alternative 3: For Biosolids Treated in Other Processes. Demonstrate that the process can reduce enteric viruses and viable helminth egg ova. Maintain operating conditions used in the demonstration.	Alternative 3: Use of Processes Equivalent to PSRP. Biosolids are treated in a process equivalent to one of the PSRPs, as determined by the permitting authority.
Alternative 4: Biosolids Treated in Unknown Processes. Demonstration of the process is unnecessary. Instead, test for pathogens Salmonella sp. or fecal coliform bacteria, enteric viruses, and viable helminth ova--at the time the biosolids are used or disposed of, or are prepared for sale or giveaway.	--
Alternative 5: Use of Further Reduce Pathogens (PFRP). Biosolids are treated in one of the PFRP identified in 40 CFR Part 503.	--
Alternative 6: Use of a Process Equivalent to PFRP. Biosolids are treated in a process equivalent to one of the PFRPs, as determined by the permitting authority.	--
1. Source: EPA, September 1995 2. Class A Biosolids are biosolids that contain no detectable level of pathogens. 3. Class B Biosolids are biosolids that are treated but still contain a detectable level of pathogens. 4. CFR: Code of Federal Regulations	

All land-applied biosolids must also comply with one of the applicable vector attraction reduction requirements specified in 40 CFR 503.33. Table 5-6 summarizes the vector attraction reduction options identified in 40 CFR Part 503.

<b>Table 5-6</b> <b>Vector Attraction Reduction Options<sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>	
<b>Option Number</b>	<b>Description of Option</b>
1	Reduce the mass of volatile solids by a minimum of 38%.
2	Demonstrate vector attraction reduction with additional anaerobic digestion in a bench-scale unit.
3	Demonstrate vector attraction reduction with additional aerobic digestion in a bench-scale unit.
4	Meet a specific oxygen demand uptake rate for aerobically treated biosolids.
5	Use aerobic processes at an average temperature of 40°C for 14 days or longer.
6	Add alkaline materials to raise the pH under specified conditions.
7	Reduce moisture content of biosolids that do not contain unstabilized solids from other than primary treatment to at least 75% solids.
8	Reduce moisture content of biosolids with unstabilized solids to at least 90%.
9	Inject biosolids beneath the soil surface within a specified time, depending on the level of pathogen treatment.
10	Incorporate biosolids applied to or placed on the land surface within specified periods after application to or placement on the land surface.
1. Source: EPA 40 CFR Part 503: Biosolids Rule, Land Application	

In addition to the pathogen reduction and vector attraction reduction requirements, the following standards also apply for land application of biosolids:

- Biosolids application rates must not exceed the nitrogen agronomic rates of the crop being planted.
- A biosolid with a moisture content of less than 75% shall not be applied during periods when wind speeds exceed 25 miles per hour.
- Biosolids are not to be applied in amounts exceeding the Risk Assessment Acceptable Soil Concentration as described by the following equation:

$$BC = RP - 1.8 (BS)$$

where:

BC = Background Cumulative Adjusted Loading Rate (pounds per acre [lbs/acre])  
 RP = 40 CFR Part 503 Cumulative Pollutant Loading Rate (lbs/acre)  
 BS = Actual Site Background Site Soil Concentration (milligrams per kilogram [mg/kg])

Table 5-7 summarizes 40 CFR Part 503 pollutant limits.



<b>Table 5-7</b> <b>Pollutant Limits for Land Applied Biosolids<sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>				
<b>Constituent</b>	<b>Maximum Value in all Biosolids (mg/kg)<sup>2</sup></b>	<b>Maximum Value in EQ<sup>3</sup> and PC<sup>4</sup> Biosolids (mg/kg)</b>	<b>Annual Loading Rate (kg/ha)<sup>3</sup></b>	<b>Lifetime Loading Rate (kg/ha)</b>
Arsenic	75	41	2.0	41
Cadmium	85	39	1.9	39
Chromium	3,000	1,200	150	3,000
Copper	4,300	1,500	75	1,500
Lead	840	300	15	300
Mercury	57	17	0.85	17
Molybdenum	75	18	0.90	18
Nickel	40	420	21	420
Selenium	100	36	5.0	100
Zinc	7,500	2,800	140	2,800
1. Source: EPA, September 1995 2. mg/kg: milligram per kilogram 3. EQ: Excellent Quality biosolids, as defined in 40 CFR Part 503 4. PC: Pollutant Concentration biosolids, as defined in 40 CFR Part 503 5. kg/ha: kilogram per hectare				

In addition to the pollutant limits, land-applied biosolids must also meet the following requirements:

- Biosolids to be tilled into the soil must be incorporated into the soil within 48 hours in non-arid areas during the period from May 1 through September 30.
- Grazing of domesticated animals in areas where biosolids have been applied is restricted until the necessary waiting period has elapsed.
- Application of biosolids to slopes of greater than 10% requires an erosion control plan.
- Tail water (water located immediately downstream) from conveying structures shall be designed and maintained to minimize field erosion.
- Staging and biosolids application areas must be at least:
  - 10 feet from property lines;
  - 500 feet from domestic water supply wells;
  - 100 feet from non-domestic water supply wells;
  - 50 feet from public roads and occupied on-site residences;
  - 100 feet from surface waters, including wetlands, creeks, ponds, lakes, underground aqueducts, and marshes;
  - 33 feet from primary agricultural drainages;
  - 500 feet from occupied non-agricultural buildings and off-site residences;
  - 400 feet from a domestic water supply reservoir;

- 200 feet from primary tributary to a domestic water supply;
- 2,500 feet from any domestic surface water supply intake; and
- 500 feet from enclosed water bodies that could be occupied by pupfish.

## **5.2 Basis of Design**

The evaluation of alternatives presented in Section 7 includes the design criteria evaluation for each WWMF treatment system component. Projected wastewater flows used for the capacity analysis and preliminary design were described in Section 3.

## **5.3 Basis for Cost Estimates**

The estimated construction costs included in the evaluation of alternatives as presented in Section 7 are based on actual construction bidding results from similar work, published cost guides, and other construction cost experience. Reference was made to the available drawings of the existing facilities to determine construction quantities. Where required, estimates were based on preliminary layouts of the proposed improvements.

### **5.3.1 Contingencies**

A contingency factor equal to 20% of the estimated construction cost has been added. Recognizing that the cost estimates are based on concept design, allowances must be made for variations in final quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigations, and other difficulties that cannot be foreseen at this time, but that may tend to increase final costs.

### **5.3.2 Engineering**

The cost of engineering services for major projects typically include special investigations, a pre-design report, surveying, foundation exploration, preparation of contract drawings and specifications, bidding services, construction management, inspection, construction staking, start-up services, and the preparation of operation and maintenance manuals. Depending on the size and type of the project, engineering costs may range from 15 to 25% of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small, complicated projects. The engineering costs for design and construction of the proposed project will average about 20% of the construction cost. An additional 5% was added to the average engineering costs to account for anticipated project planning and permitting needs, resulting in the 25% engineering cost used for the cost estimates presented in this study.

### **5.3.3 Legal and Administrative**

An allowance of 4% of construction cost has been added for legal and administrative services. This allowance is intended to include internal project planning and budgeting, project administration, liaison, interest on interim financing, legal services, review fees, legal advertising, and other related expenses associated with the project.

## 5.4 Basis for Alternatives Considered

In February 2010, the District and SHN initiated efforts to develop a feasibility study that identified the most feasible options for future upgrades to the MCSD wastewater treatment, reclamation, and disposal facilities. The goal of the feasibility study process was to identify the final alternatives to be considered in the 20-year facilities plan.

A series of public workshops, presentations, and technical review sessions were held in 2010 as part of the feasibility study process. A public scoping workshop was held in April 2010 to engage interested community members. The workshop presented an opportunity for rate payers and stakeholders to provide input on alternatives to be considered. The outcome of the workshop included a list of ideas and treatment system goals from the public to be considered. A technical review session was then held in June 2010. During this workshop, SHN and District staff selected the final evaluation criteria, weighting factors, and scoring system for the process evaluation. The group then developed a process option evaluation matrix, which involved applying criteria and ranking ideas for all process options identified. The outcome of the technical review session included a final ranking of alternatives to be considered in the 20-year facilities plan.

An update on the feasibility study was presented to the MCSD Board in July 2010 and included a request for the Board to approve the top four alternatives from the feasibility study for further review in the 20-year facilities plan. Board approval was granted with slight modifications to the alternatives to be considered. The top four treatment alternatives approved for review included an expanded lagoon/wetland system, suspended aeration system, oxidation ditch, and membrane treatment system. The Board also approved further review of the ocean outfall disposal alternative as part of the 20-year facilities planning process. Following Board approval in July 2010, the facilities plan was completed using the top four treatment alternatives as identified during the feasibility study process. Additional scoping efforts were also initiated regarding the ocean outfall disposal alternative; further discussion regarding this process is included in Section 8.4.

Based on the peer review comments provided by Kennedy-Jenks in August 2011, the District requested that one additional treatment alternative be added to the facilities plan. Based on this request, a conventional activated sludge system was added to the treatment alternatives reviewed in the facilities plan.



## 6.0 Collection System Analysis

### 6.1 Model Description

#### 6.1.1 Model Development

The MCSD collection system model was initially developed using the program StormNet, a proprietary software package distributed by Boss International. Over the course of the model development, the program was transferred from Boss to Autodesk and renamed the Storm and Sanitary Analysis program. The 2011 version of the Autodesk Storm and Sanitary Analysis software was used for the analysis presented in this report.

The purpose of the collection system model was to create a tool that can be used to evaluate the MCSD collection system under existing and projected flow conditions. The model is still under development as of the publish date for this facilities plan and will continue to be calibrated with new data as it becomes available during the next wet weather season. The model is currently being used as preliminary analysis tool to determine if there are areas in the collection system that may be limited in capacity under existing and/or varying growth conditions. The majority of the residential and commercial establishments in McKinleyville are located east of Highway 101 and the treatment facility is located west of Highway 101. Figure 1-5, presented in Section 1, shows the extent of the collection system for the MCSD service area.

#### 6.1.2 Collection System Data

MCSD maintains a GIS database of its collection system. The GIS database provided the base layer of information for the collection system model. The pipes in the database were converted to links in the model and the manholes and cleanouts were converted to junctions. Each link had invert elevations assigned and a sanitary time pattern was applied at each junction. Main line pipes 6-inches and larger were included in the model. The distribution of pipe size as modeled for the MCSD collection system is shown in Table 6-1.

<b>Table 6-1</b> <b>Pipe Size Distribution<sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>		
<b>Pipe Diameter</b>	<b>Total Number of Pipe Segments</b>	<b>Total Length (feet)</b>
6 inch	957	255,707
8 inch	92	28,327
10 inch	44	15,634
12 inch	36	10,371
15 inch	40	11,296
16 inch	1	106
18 inch	11	2,802
21 inch	4	1,702
24 inch	23	8,700
1. 4-inch and 2-inch pipes were not evaluated in the collection system model.		

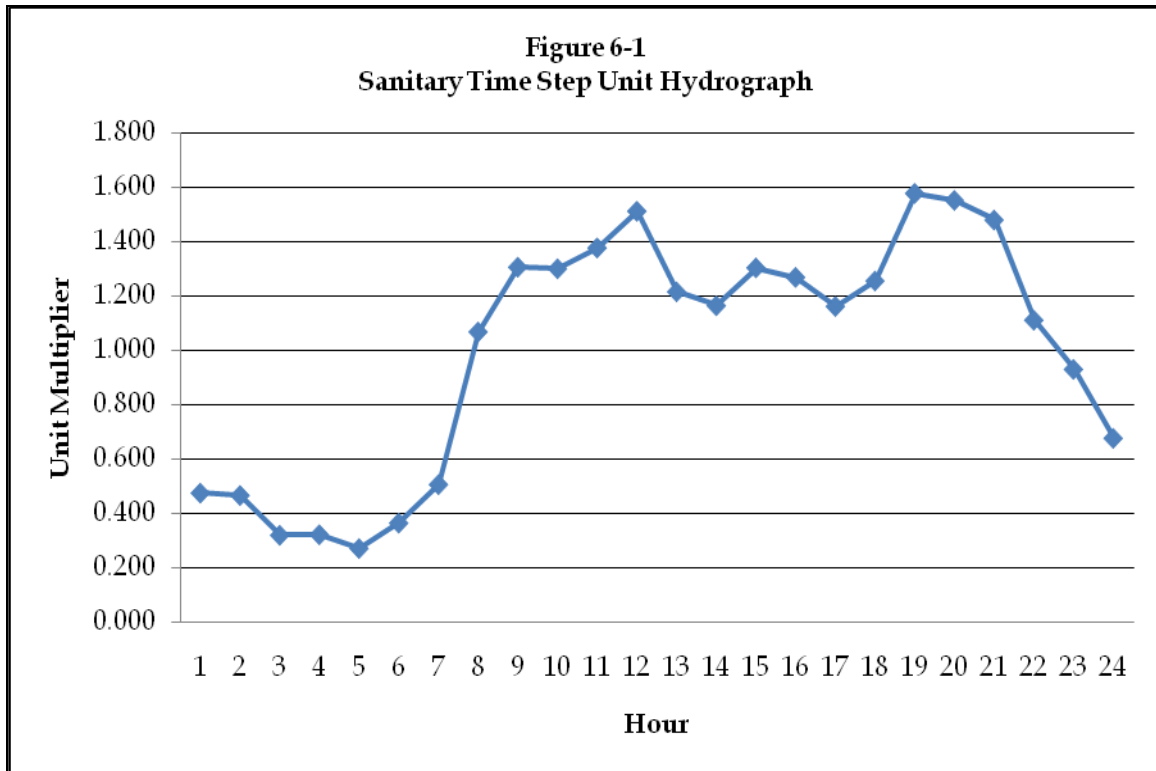
### 6.1.3 Pump Station Data

The pump data for each lift station was based on information provided by the District and pump drawdown tests conducted in 2009 and 2011. Table 6-2 summarizes the pumping capacity used in the model for each lift station.

Table 6-2 Lift Station Pump Capacity Summary MCSD Wastewater Management Facility					
Name	Pump #	Description	Design Capacity <sup>1</sup> (GPM) <sup>2</sup>	Test Flow <sup>3</sup> (GPM)	Firm Capacity <sup>4</sup> (GPM)
B Street (PS #1) <sup>5</sup>	1	Pump 1 only	225	192	182
	2	Pump 2 only	225	182	
	1, 2	Pump 1 and 2	---	242	
Letz (PS #2)	1	Pump 1 only	500	316	673
	2	Pump 2 only	500	219	
	1, 2	Pump 1 and 2	---	673	
	3	Pump 3 only	1,100	782	
Kelly (PS #3)	1	Pump 1 only	225	125	125
	2	Pump 2 only	225	131	
	1, 2	Pump 1 and 2	---	---	
Hiller (PS #4)	1	Pump 1 only	550	836	836
	2	Pump 2 only	550	836	
	2	Pump 1 and 2	---	1,338	
Fisher (PS #5)	1	Pump 1 only	900	575	1,614
	2	Pump 2 only	900	613	
	1, 2	Pump 1 and 2	---	924	
	3	Pump 3 only	1,500	1,614	
	4	Pump 4 only	1,500	1,643	
1. Operating point indicated on pump curves 2. GPM: gallons per minute 3. As determined during drawdown tests conducted in 2009 and 2011 4. Firm capacity assumes largest pump is offline 5. PS: pump station					

### 6.1.4 Sanitary Time Pattern Development

A sanitary time pattern was developed for the collection system based on dry weather flow monitoring conducted in July 2011. Flows were recorded daily in 15 minute intervals and then averaged over an hour. The location for the flow monitoring was chosen to avoid influence from tributary pump stations. The sanitary time step unit hydrograph developed from the flow metering data is shown in Figure 6-1.



### 6.1.5 RDII Input Parameters

The storm and sanitary sewer analysis model allows users to integrate Rainfall Derived Infiltration and Inflow (RDII) into the sanitary sewer analysis. The model uses a RDII unit hydrograph (UH) to estimate the amount of rainfall that enters the collection system. The RDII unit hydrograph is defined by three parameters:

- R, the percentage of rainfall that enters the sanitary sewer analysis system as RDII
- T, the time from the onset of rainfall to the peak of the UH in hours
- K, the ration of time to recession of the UH to the time to peak

For the MCSD collection system analysis, RDII input parameters were estimated based on recorded flows during representative 2 and 5-year 24-hour rainfall events. For purposes of the RDII analysis the default hydrologic parameters associated with the time series data for Humboldt County (Arcata) was used. The 2-year intensity storm shown for Arcata had a total rainfall amount of 3.5 inches and the 5-year intensity storm had a total rainfall amount of 4.5 inches.

Table 3-3 (in Section 3) provides a summary of the anticipated total influent flow associated with varying rainfall depths. Table 6-3 shows a summary of the wet weather flow allocation for existing conditions based on the RDII analysis.



<b>Table 6-3</b> <b>Summary of Collection System Model Wet Weather Flow Allocation</b> <b>MCSD Wastewater Management Facility</b>			
<b>Return Interval</b>	<b>Rainfall (inches)</b>	<b>Rainfall Derived Infiltration and Inflow<sup>1</sup> (MGD)<sup>2</sup></b>	<b>Total Flow (MGD)</b>
None	0	0.00	0.95
2-Year	3.5	0.84	1.79
5-Year	4.5	1.10	2.05 <sup>1</sup>
100-Year	6.9	1.72	2.67
1. Corresponds to the peak day average flow shown in Table 3-4 2. MGD: million gallons per day			

## 6.2 Model Simulations

### 6.2.1 Existing Flows

Existing dry-weather flows were allocated in the collection system model based on the distribution of single family, multi-family and commercial developments in McKinleyville. Each development type was assigned an EDU allocation as shown in Table 6-4. As of December 2010, the District had approximately 4,048 single-family sewer connections. EDU allocations for the remaining multi-family and commercial developments were based on review of water use records for 2009 and 2010. An additional 93 single-family connections were included in the model analysis to account for high water use multi-family units (for example, mobile home parks). For a base flow equal to 180 gpd/EDU, the total dry-weather flow allocation included in the model is approximately 0.95 MGD.

<b>Table 6-4</b> <b>Summary of Dry Weather Flow Allocation</b> <b>MCSD Wastewater Management Facility</b>			
<b>Development Type</b>	<b>EDU<sup>1</sup> Allocation</b>	<b>EDUs</b>	<b>Total Flow<sup>2</sup> (gpd)<sup>3</sup></b>
Single Family Residential	Direct (1:1) <sup>2</sup>	4,141	745,380
Multi-Family Residential	0.56 x Number of Units <sup>4</sup>	746	134,280
Commercial	Based on 90% of water usage <sup>5</sup>	380	68,400
Total	---	5,267	948,060
1. EDU: Equivalent Dwelling Unit 2. Commercial EDUs will vary based on water use data. 3. gpd: gallons per day 4. Based on 180 gpd/EDU. 5. Based on 4,048 single-family sewer connections at year end 2010. Includes additional allocation of approximately 93 EDUs for high water use multi-family units.			

### 6.2.2 Future Development Scenarios

The District has been taking steps to identify and project the affects of growth in central McKinleyville; however, it is largely dependent on the full extent of the County's development plan

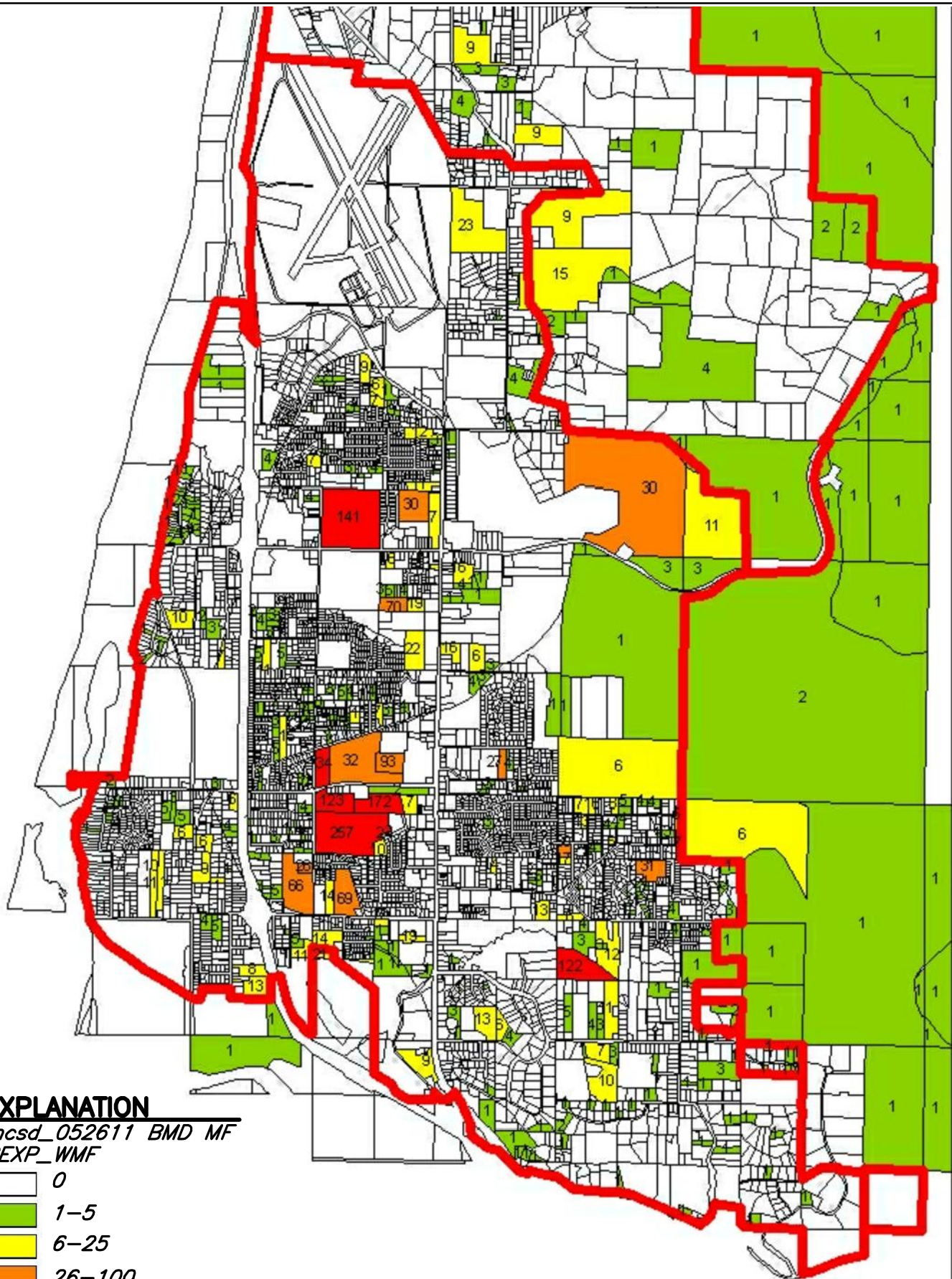
for McKinleyville. As part of the Humboldt County general plan update, the County has provided the District with a variety of new development projections. The County is also currently addressing multi-family rezoning efforts in McKinleyville, and how these efforts may change the general plan projected growth scenarios.

On May 26, 2011, the County provided the District with the latest combined development projection data set for the MCSD service area. The data set provided the estimated number of development units that would be developed in McKinleyville for varying general plan and multi – family rezone planning conditions. Table 6-5 summarizes the development unit projections provided by the County for the various growth scenarios.

<b>Table 6-5</b> <b>General Plan and Multi-Family Rezone Development Projections<sup>1</sup></b> <b>MCSD Wastewater Management Facility</b>				
<b>Name</b>	<b>Description</b>	<b>Total Units Proposed<sup>1</sup></b>	<b>Units Within Service Area<sup>2</sup></b>	<b>Anticipated Flow Increase<sup>3</sup> (gpd)<sup>4</sup></b>
BMID-MF <sup>4</sup>	GP Update/MF Mid	2,760	2,562	461,160
BMAX-MF <sup>5</sup>	GP Update/MF Max	7,222	6,898	1,241,640
DMID-MF <sup>6</sup>	GP/MF Mid	2,057	1,850	333,000
DMAX-MF <sup>7</sup>	GP/MF Max	6,291	5,965	1,073,700
1. As provided by Humboldt County on May 26, 2011. 2. Based on service area as presented in the collection system model. 3. Anticipated flow increase based on estimated contribution of 180 gpd/unit. 4. gpd: gallons per day 5. Development potential at expected density for General Plan Update Alternative B with Multi-Family (MF) rezone parcels at expected density substituted. 6. Development potential at maximum density for General Plan Update Alternative B with MF rezone parcels at maximum density substituted. 7. Development potential at expected density for General Plan Alternative D with MF rezone parcels at expected density substituted. 8. Development potential at maximum density for General Plan Alternative D with MF rezone parcels at maximum density substituted.				

As shown in Table 3-7, the projected number of EDUs for year 2030 is 7,525, which is approximately 2,260 more EDUs than the number of EDUs for 2010 (5,267). Because the growth projection for Alternative B with Multi-Family (BMID-MF) scenario includes the development of approximately 2,500 new EDUs in the District's service area, this scenario was used for evaluation of the collection system under projected flow conditions. The County-provided GIS data set included a direct allocation of development allocation by parcel. Figure 6-2 shows the representative distribution of development for the BMID-MF scenario. Table 6-6 shows the flow allocation for the BMID-MF scenario for each wet weather flow condition.

\\z:\projects\2008\008189-MCSD\300-FacilitiesPlan\Drawings . . . SAVED: 10/5/2011 1:21 PM NDOWNEY . PLOTTED: 10/6/2011 12:24 PM . NATHAN DOWNEY



# **EXPLANATION**

*mcsd\_052611 BMD MF*  
*BEXP\_WMF*

- 0
- 1-5
- 6-25
- 26-100
- 101-500

<p align="center"><b>Table 6-6</b> <b>Summary of Flow Allocations for Existing and Projected Conditions</b> <b>MCSD Wastewater Management Facility</b></p>					
<b>Return Interval</b>	<b>RDII<sup>1</sup> (MGD)<sup>2</sup></b>	<b>Average Dry Weather Flow (MGD)</b>	<b>RDII<sup>2</sup> Wet Weather Flow (MGD)</b>	<b>BMID-MF<sup>3</sup> Flow Increase (MGD)</b>	<b>BMID-MF Total Flow (MGD)</b>
None	0.00	0.95	0.95	0.46	1.41
2-Year	0.84	0.95	1.79	0.46	2.25
5-Year	1.10	0.95	2.05	0.46	2.51
100-Year	1.72	0.95	2.67	0.46	3.13
<p>1. RDII: Rainfall Derived Infiltration and Inflow 2. MGD: million gallons per day 3. BMID-MF: Alternative B with Multi-Family</p>					

## 6.3 Results

### 6.3.1 Pipe Capacity Assessment

The capacity of the collection system was evaluated by comparing the ratio between the predicted depth of flow and the diameter of each pipe segment investigated. Pipe segments that were at capacity and/or with flow ratios in excess of 0.75, indicating pipes are flowing at greater than 75% capacity, were identified as segments requiring further investigation.

Figure 6-3 shows the flow ratio distribution in the pipe network under existing conditions. Figures 6-4 and 6-5 show the flow ratio distribution in the pipe network for existing conditions including the 5-year and 100-year wet-weather RDII allocations, respectively.

Figure 6-6 shows the flow ratio distribution in the pipe network for the BMID-MF projected flow condition. Figures 6-7 and 6-8 show the flow ratio distribution in the pipe network for the BMID-MF flow projection including the 5-year and 100-year wet-weather RDII allocations, respectively.

### 6.3.2 Pump Capacity Assessment

The pump capacity at each lift station was evaluated by comparing the peak inflow under existing and projected conditions, with the firm capacity identified for each lift station. Table 6-7 shows the results of the lift station assessment for existing and projected peak flows.

### 6.3.3 Identified Deficiencies

As shown on Figures 6-3 through 6-8, there are three gravity collection trunk lines that extend under Highway 101 conveying the majority of the wastewater flows from the east side to the west side of McKinleyville where the treatment and disposal facilities are located. For the existing dry-weather conditions (Figure 6-3), and up to the existing 5-year RDII condition (Figure 6-4), the collection system is able to collect and convey wastewater flows through the pipe network without surcharging. Under the existing 100-year RDII scenario (Figure 6-5), the system shows some surcharging in the main trunk lines (Lines 3 and 5). For the BMID-MF projected dry-weather flow



## **EXPLANATION**

### **PIPE-EXISTING**

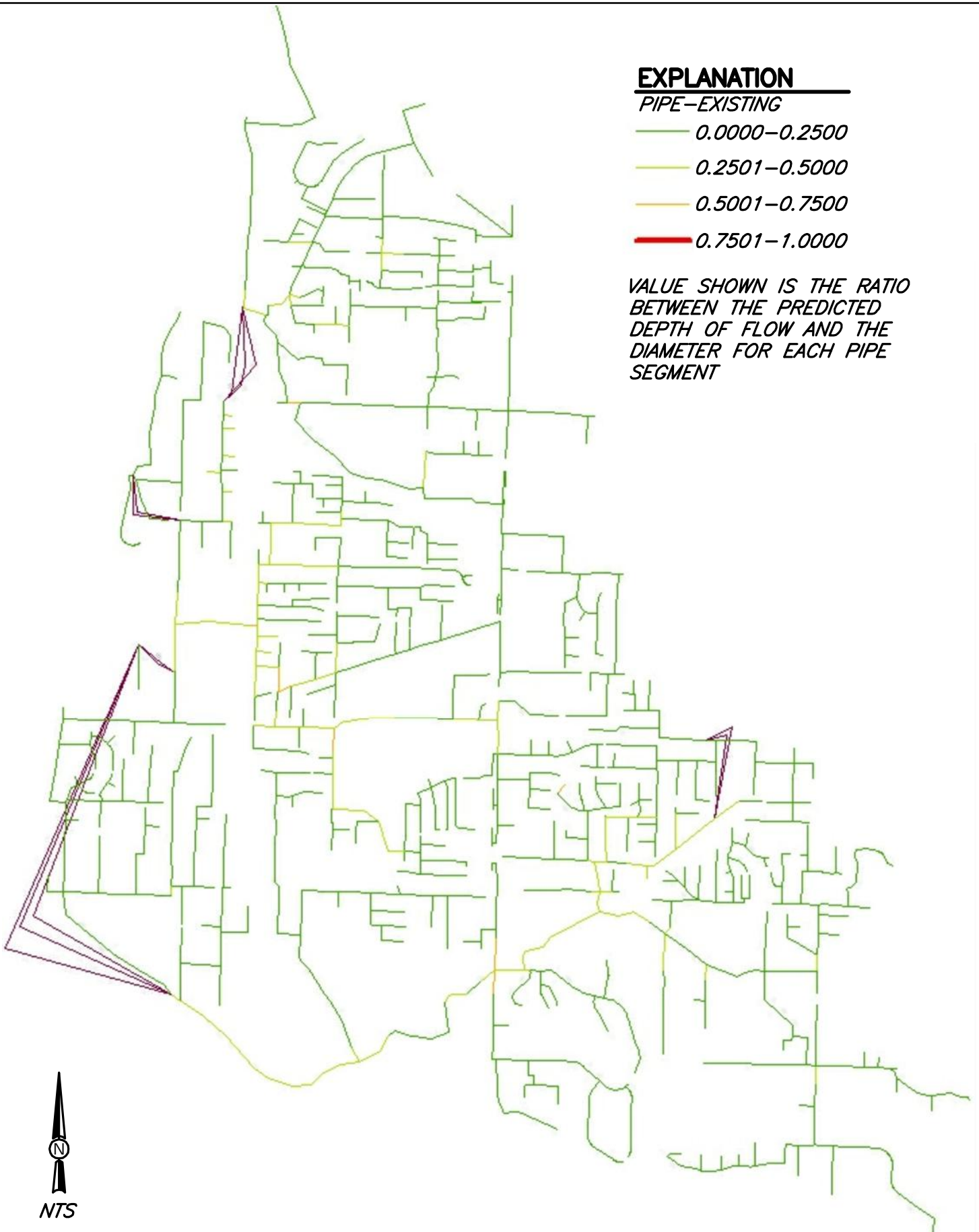
0.0000-0.2500

0.2501-0.5000

0.5001-0.7500

0.7501-1.0000

VALUE SHOWN IS THE RATIO  
BETWEEN THE PREDICTED  
DEPTH OF FLOW AND THE  
DIAMETER FOR EACH PIPE  
SEGMENT



NTS



## EXPLANATION

PIPE-EXISTING 5 YR

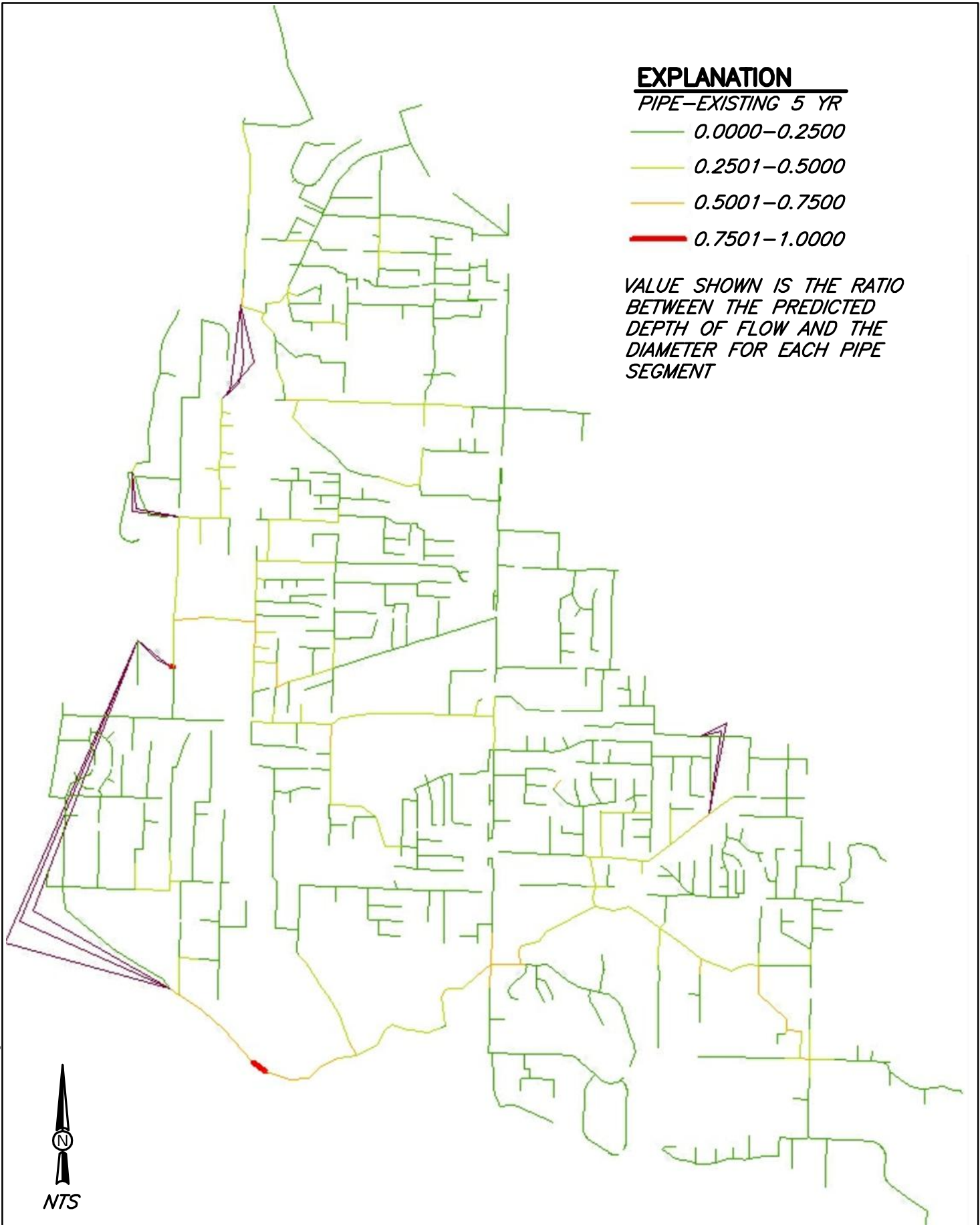
0.0000-0.2500

0.2501-0.5000

0.5001-0.7500

0.7501-1.0000

VALUE SHOWN IS THE RATIO  
BETWEEN THE PREDICTED  
DEPTH OF FLOW AND THE  
DIAMETER FOR EACH PIPE  
SEGMENT



NTS

## EXPLANATION

PIPE-EXISTING 100 YR

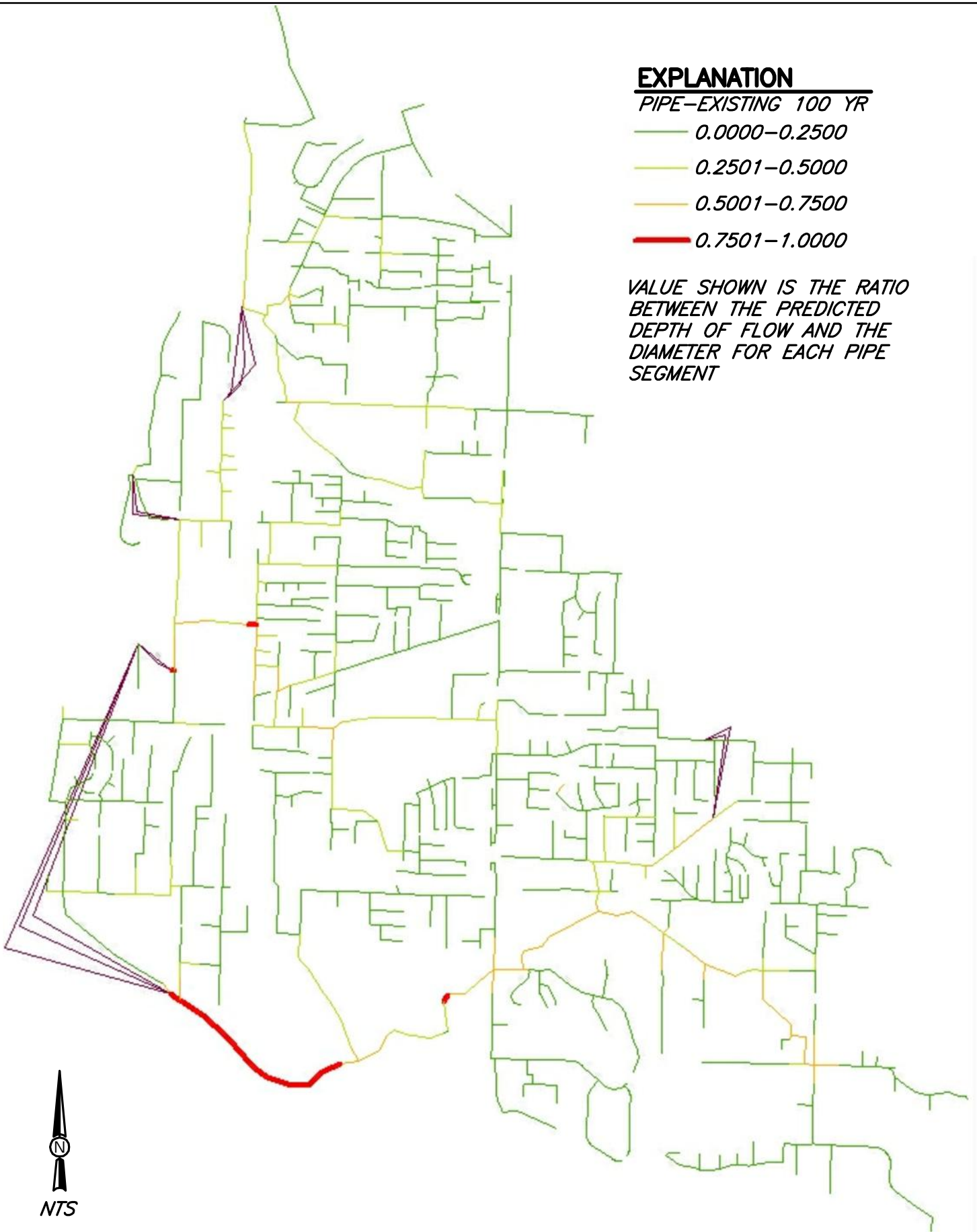
0.0000-0.2500

0.2501-0.5000

0.5001-0.7500

0.7501-1.0000

VALUE SHOWN IS THE RATIO  
BETWEEN THE PREDICTED  
DEPTH OF FLOW AND THE  
DIAMETER FOR EACH PIPE  
SEGMENT



NTS

## EXPLANATION

PIPE-BMID MF

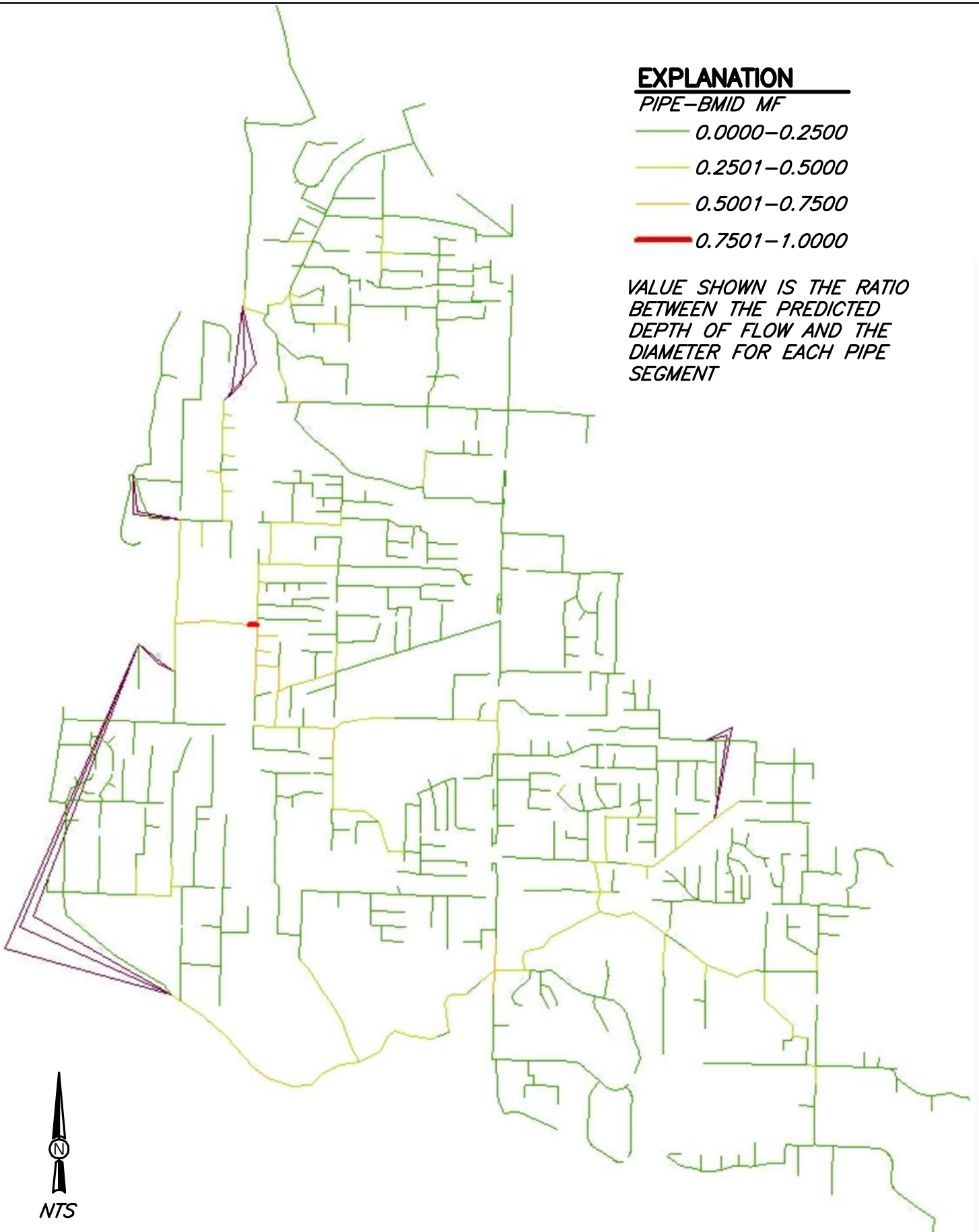
0.0000-0.2500

0.2501-0.5000

0.5001-0.7500

0.7501-1.0000

VALUE SHOWN IS THE RATIO  
BETWEEN THE PREDICTED  
DEPTH OF FLOW AND THE  
DIAMETER FOR EACH PIPE  
SEGMENT



NTS

## EXPLANATION

PIPE-BMID MF 5 YR

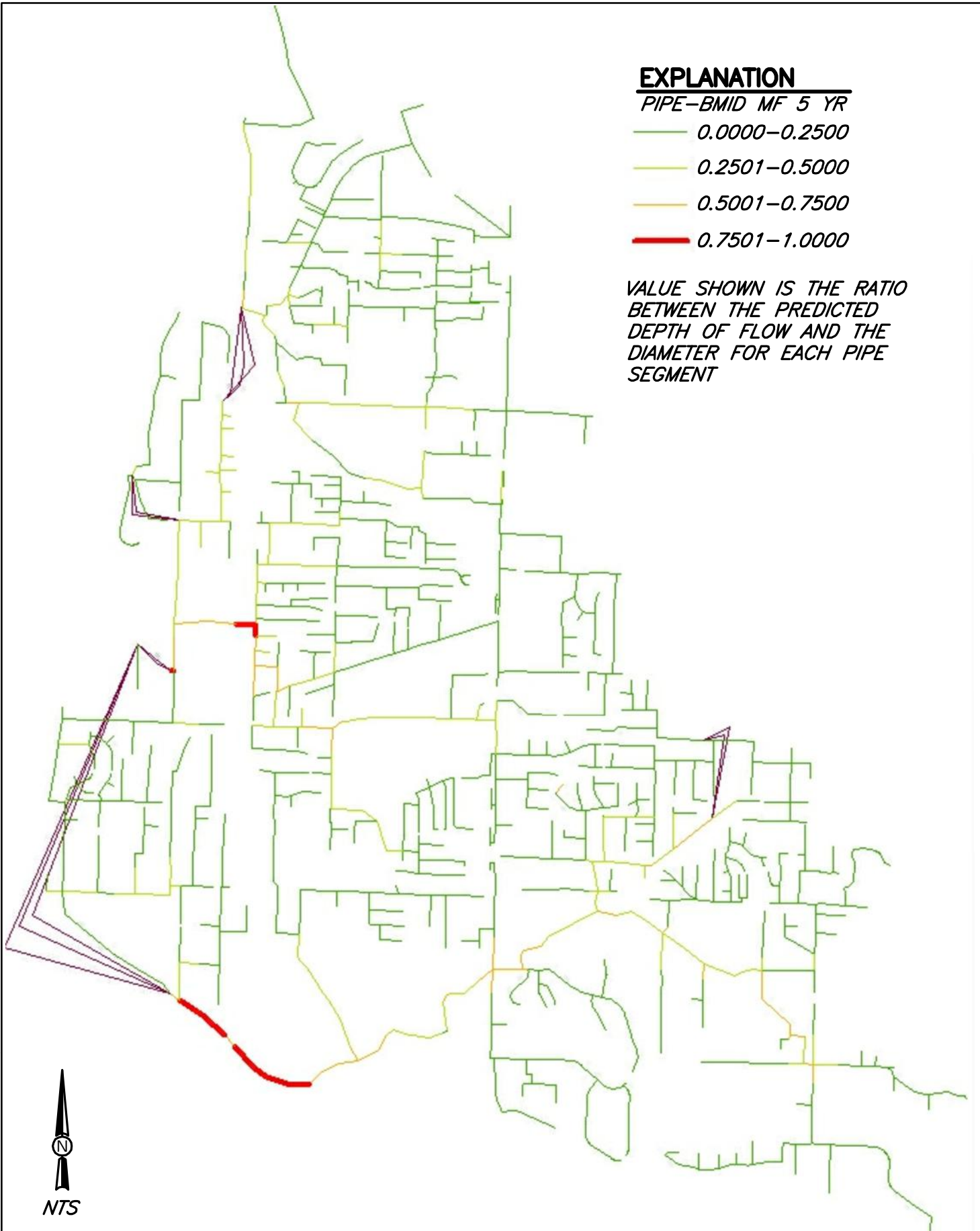
0.0000-0.2500

0.2501-0.5000

0.5001-0.7500

0.7501-1.0000

VALUE SHOWN IS THE RATIO  
BETWEEN THE PREDICTED  
DEPTH OF FLOW AND THE  
DIAMETER FOR EACH PIPE  
SEGMENT



NTS



## EXPLANATION

PIPE-BMID MF 100 YR

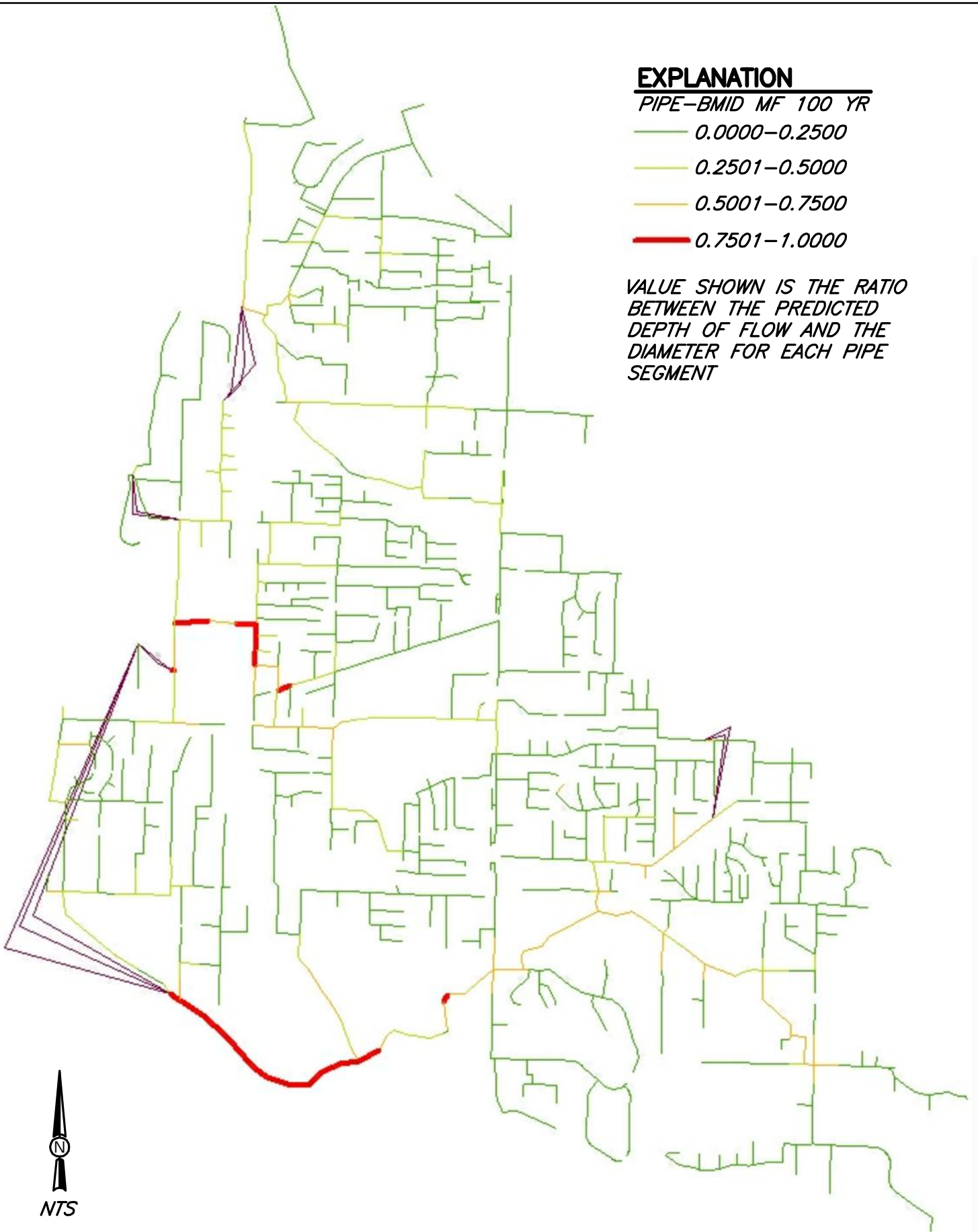
0.0000-0.2500

0.2501-0.5000

0.5001-0.7500

0.7501-1.0000

VALUE SHOWN IS THE RATIO  
BETWEEN THE PREDICTED  
DEPTH OF FLOW AND THE  
DIAMETER FOR EACH PIPE  
SEGMENT



NTS



conditions (Figure 6-6), the system shows some surcharging, and under the 5-year and 100-year RDII conditions (Figures 6-7 and 6-8, respectively) the system shows extensive surcharging in the main trunk lines.

<b>Table 6-7</b> <b>Lift Station Assessment at Peak Flows</b> <b>MCSD Wastewater Management Facility</b>				
<b>Lift Station</b>	<b>Firm Capacity (gpm)<sup>1</sup></b>	<b>Existing Peak Instantaneous Flow<sup>2</sup> (gpm)</b>	<b>Projected Peak Instantaneous Flow<sup>3</sup> (gpm)</b>	<b>Exceed Firm Capacity (Y/N)</b>
B Street (PS #1)	182	120	146	N
Letz Lane (PS #2)	673	814	974	Y
Kelly (PS #3)	125	86	93	N
Hiller (PS #4)	836	954	1,000	Y
Fisher (PS #5)	1,614	1,895	2,506	Y
1. gpm: gallons per minute 2. Based on total existing peak instantaneous flow of 2.67 MGD 3. Based on total projected peak instantaneous flow of 3.13 MGD				

The results of the lift station pump assessment show that the Letz, Hiller, and Fisher lift stations are limited in their capacity to handle existing and projected peak flows based on firm capacity alone.

## 7.0 Treatment Alternatives

Current organic and suspended solids loadings on the MCSD facultative pond system with supplemental aeration are at the high range of the system's capacity to provide adequate secondary treatment. Secondary treatment alternatives are presented for providing improved performance, reliability, and the additional treatment capacity to meet future loadings. The secondary treatment alternatives included for review in this section were selected based on the WWMF feasibility study planning efforts undertaken in 2010 by the District and SHN.

Treatment alternatives must also address nitrogen removal. In addition to concerns about ammonia toxicity violations, compliance with surface water discharge limitations requires effluent nitrate-nitrogen concentrations to be less than 10 mg/L. Because land reclamation is based on agronomic application rates, effluent nitrogen concentrations are also limited by the land available for irrigation of effluent during the non-discharge period. This section includes an evaluation of treatment systems for the ability to reduce total nitrogen to less than 10 mg/L. This is a common land application standard – one that will increase the facility's disposal options.

Options for addressing system deficiencies not directly related to secondary performance are also evaluated in this section. The lack of any pre-treatment facilities for screening and grit removal is one such deficiency. An evaluation of pre-screening and grit removal alternatives precedes the discussion of secondary treatment alternatives. Discussion of auxiliary systems, such as biosolids management, is included in the discussion of secondary treatment options. We have also included a detailed evaluation of options for biosolids stabilization, storage, and disposal for the preferred secondary treatment option.

### 7.1 New Headworks

Wastewater contains large solids that can interfere with treatment processes or cause undue mechanical wear and increased maintenance on wastewater treatment equipment. To minimize potential problems, these materials are removed from the influent wastewater. Preliminary treatment at the MCSD facility includes pre-screening, and grit removal.

#### 7.1.1 Pre-Screening

Screening is especially important in treatment systems without primary clarifiers. Fine screens have openings from  $1/16$  to  $1/4$  -inch in diameter and are used to remove material that may significantly impair downstream solid and liquid processes. It is important to remove inorganic solid material because it will contribute to the formation of a scum layer in downstream basins, clarifiers, tertiary wetlands, or contact basins. Unscreened wastewater also damages pumping equipment, and can contribute to plugging of diffusers in aeration basin(s). Plastics and other solids that make it through the treatment process cause problems for biosolids and effluent disposal. Types of fine screen equipment include:

- Inclined drum or cylindrical screen
- Belt screen–continuous self cleaning
- Band screen with center feed

#### 7.1.1.1 Inclined Spiral Screen

Drum or cylindrical screens are placed in a channel at an incline in the direction of flow. Wastewater flows into the interior of the drum and through the screen. Solids are scraped off of the interior of the screen mechanically with either brushes or a small rake depending on the manufacturer. From the interior of the cylindrical screen an auger conveys solids up an incline of approximately 35 degrees. The solids material is compressed and dewatered as it moves up the conveyor and is then dropped into a dumpster.

##### Advantages:

- Less expensive than continuous self-cleaning screens
- Provides dewatering and compaction in the screw conveyor
- Designed for lower flows
- Low head loss

##### Disadvantages:

- Longer channel (inclined at angle of 35 degrees)

#### 7.1.1.2 Belt Screens

Continuous self-cleaning screens consist of a continuous “belt” of plastic or stainless steel elements installed at a 70 degree angle in the channel and pulled through wastewater to provide screening. Screen openings range from  $\frac{1}{8}$  to  $\frac{1}{4}$  inch in diameter. The continuous screening action of these screens allows efficient removal of large quantities of solids and because of the greater solids handling capacity, smaller openings may be used.

##### Advantages:

- Very efficient
- Shorter channel length (inclined at 70 degrees to horizontal)

##### Disadvantages:

- Lower range of flows than drum screen
- More expensive
- Greater head loss
- Screenings compactor required downstream of unit

#### 7.1.1.3 Band Screens

Band screens are similar to belt screens but have a vertical configuration parallel to the channel walls. With a center feed type screen, wastewater flows out through both sides of the screen, doubling the screening area. The wastewater enters through the center of the screen, passes through the stainless steel sieve elements and exits through the sides of the unit. Screenings are captured by the elements and are carried up to the discharge point where they are removed by a pressurized spray header system. The material is discharged into a sluice trough and conveyed for collection and disposal, typically to a screenings washing and dewatering system.

#### 7.1.1.4 Design Criteria

Because of the lack of primary treatment at the WWMF it is recommended that some degree of fine screening, with a maximum screen opening of 0.25 inches, be provided. All of the fine screening units discussed in the previous section normally are placed following the pump station because screens with openings less than 0.50 inches in diameter will collect too much fecal material when placed prior to a pump station in a gravity collection system.

Some wastewater treatment systems use both coarse screening (greater than ½-inch) in the form of bar screen placed prior to the pumps and fine screening placed after the pumps. This is not considered the best option for the MCSD wastewater treatment system where solids loading does not warrant installation of both types of screens.

Pre-screening could be accomplished by one screening unit designed to handle the projected (2030) PIF of 3.8 MGD. Flows greater than this would be diverted to a bypass channel equipped with coarse screening in the form of a bar screen. However for most screens, the design of the approach channel is limiting at low flows because minimum velocities cannot be maintained in a channel sized for screens rated for the larger flow. Generally the screening channel should be designed to maintain a 1.25 feet per second (ft/s) minimum velocity at peak hourly flows during dry weather conditions to prevent solid material from settling out ahead of the screen. To maintain this velocity at the current ADWF of 1.0 MGD, the channel should not be greater than 2 feet in width.

#### 7.1.1.5 Summary of Options

Screening options for the WWMF include either a single screen designed for a projected PIF of 3.8 MGD or two channels, each provided with a 2-MGD screen. Equipment costs for two spiral screens are approximately \$130,000. Equipment costs for two belt or band screens with redundant compactors are estimated to be \$275,000. Generally installation of belt or band screens is also more expensive because of the increased channel depth required and the need to provide for more conveyance and compaction of screened material. The least expensive equipment option would be to install two inclined 2 MGD spiral screens.

### 7.1.2 Grit Removal

Grit includes sand, gravel, cinder, or other heavy solid materials that are “heavier” (higher specific gravity) than the organic biodegradable solids in the wastewater. Grit also includes eggshells, bone chips, seeds, coffee grounds, and large organic particles, such as food waste. Removal of grit prevents unnecessary abrasion and wear of mechanical equipment, grit deposition in pipelines and channels, and accumulation of grit in ponds and/or aeration basins. Grit removal facilities typically follow screening to prevent large solids from interfering with grit handling equipment. In secondary treatment plants without primary clarification, grit removal should precede aeration (Metcalf & Eddy, 2003).

Grit is traditionally defined as particles larger than 0.008 inch in diameter (65 mesh) and with a specific gravity of greater than 2.65 (EPA, 1987). Equipment design was traditionally based on

removal of 95% of these particles. However, with the recent recognition that smaller particles must be removed to avoid damaging downstream processes, many modern grit removal designs are capable of removing up to 75% or more of 0.006 inch (100 mesh) material.

The following types of grit removal systems were evaluated as options for the proposed new headworks at the MCSD WWMF:

- Aerated grit chamber
- Vortex type grit chamber
- Head cell

#### **7.1.2.1 Aerated Grit Chamber**

In aerated grit chambers, grit is removed by causing the wastewater to flow in a spiral pattern. Air is introduced in the grit chamber along one side, causing a perpendicular spiral velocity pattern to flow through the tank. Heavier particles are accelerated and diverge from the streamlines, dropping to the bottom of the tank, while lighter organic particles are suspended and eventually carried out of the tank.

Aerated grit chambers use a sloped tank bottom in which the air roll pattern sweeps grit along the bottom to the low side of the chamber. A horizontal screw conveyor typically is used to convey settled grit to a hopper at the head of the tank. Once removed from the chamber, grit is usually washed with a hydrocyclone and dewatered in a grit classifier to ease handling and remove organic material. The grit is then conveyed directly to a truck, dumpster, or storage hopper. From there, the grit is taken to a landfill or other disposal facility.

Aerated grit chambers typically are designed to remove particles of 70 mesh (0.008-in) or larger, with a detention period of two to five minutes at peak hourly flow. When wastewater flows into the grit chamber, particles settle to the bottom according to their size, specific gravity, and the velocity of roll in the tank. A velocity that is too high will result in lower grit removal efficiencies, while a velocity that is too low will result in increased removal of organic materials. Proper adjustment of air velocity will result in nearly 100% removal of the desired particle size and well-washed grit.

Advantages:

- Consistent removal efficiency over a wide flow range.
- A relatively low putrescible organic content may be removed with a well-controlled rate of aeration.
- Performance of downstream units may be improved by using pre-aeration to reduce septic conditions in incoming wastewater.
- Aerated grit chambers are versatile, allowing for chemical addition, mixing, pre-aeration, and flocculation.

Disadvantages:

- Potentially harmful volatile organics and odors may be released from the aerated grit chamber, requiring odor control.



- Aerated grit chambers require more power than other grit removal processes.
- Maintenance and control of the aeration system requires additional labor.
- Size of the tank to provide adequate detention times may be prohibitive at peak flows.

Some manufacturers provide aerated grit channels as part of a combination unit manufactured in conjunction with a screenings unit. A grit conveyor in the bottom of the channel moves grit to a lateral sump from which it is transported by an inclined grit transport screw undergoing dewatering in the process. To provide a minimum of two minutes of detention time at the projected PIF of 3.8 MGD, would require two channels each 10 feet deep, 4 feet wide, and 9 feet long.

#### 7.1.2.2 Vortex-Type Grit System

The vortex-type or grit removal system relies on a mechanically induced vortex to capture grit solids in the center hopper of a circular tank. The vortex is produced by a combination of an inlet flume, sloped baffle, and adjustable rotating paddles at the center of the flume. In some systems, the vortex circulation pattern is maintained by pumps instead of paddles.

Grit settles by gravity into the bottom of the tank (in a grit hopper) while effluent exits at the top of the tank. The grit that settles into the grit hopper may be removed by a centrifugal grit pump or an airlift pump, and is usually washed and dewatered with a cyclone/classifier degritting system prior to disposal.

Flow into a vortex-type grit system should be straight, smooth, and streamlined. The straight inlet channel length typically is seven times the width of the inlet channel, or 15 feet, whichever is greater. The ideal velocity range in the influent typically is 2 to 3 ft/s at 40 to 80% of peak flow. A minimum velocity of 0.5 ft/s should be maintained at all times because lower velocities will not carry grit into the grit chamber (WEF, 1998).

##### Advantages:

- These systems remove a high percentage of fine grit, up to 73% of 140-mesh (0.004 in diameter) size.
- Vortex grit removal systems are consistently efficient over a wide flow range.
- There are no submerged bearings or parts that require maintenance.
- The footprint (horizontal dimension) of a vortex grit removal system is small relative to other grit removal systems, making it advantageous when space is an issue.
- Head loss through a vortex system is minimal, typically 6 millimeters (mm) (0.25 in). These systems are also energy efficient.

##### Disadvantages:

- Vortex grit removal systems usually are of a proprietary design, which makes modifications difficult.
- Paddles may collect material not removed by the screening equipment.

- Vortex units usually require deep excavation due to their depth, which increases construction costs, especially if unrippable rock is present.
- The grit sump tends to clog and requires high-pressure agitation using water or air to loosen grit compacted in the sump.

The vortex grit removal system and the head cell provide more efficient grit removal than the aerated grit chamber and remove smaller material. A vortex grit chamber rated for 3.8 MGD would be approximately 12 feet deep, 12 feet in diameter at the top and 4 feet in diameter at the bottom.

#### 7.1.2.3 HeadCell®

A HeadCell® is a modular, multi-tray solids concentrator that removes grit down to 50 micron in size (200 mesh). Grit removal in the system is based on creating a vortex action across multiple tray layers that serve as settling basins (similar to settling tubes in clarifiers). Solids caught in the vortex flow are swept down each tray to a central core where they are collected and pumped to a grit washing device. The use of stacked multiple trays creates a large surface area that effectively captures grit in a relatively small footprint.

A HeadCell® unit capable of a 95% grit removal efficiency at the PIF of 3.8 MGD process design flow would be 6 feet in diameter, and would be placed in the center of a surrounding concrete vessel. The vessel would have a 2-foot wide circular channel extending around the exterior circumference of the head cell unit. The bottom and sidewalls of the tank would be sloped down with an overflow weir installed across the concrete containment vessel.

##### Advantages:

- High removal efficiency
- Low head loss
- Relatively small footprint
- Relatively low equipment and construction costs
- Passive operation, no moving parts

##### Disadvantages:

- Proprietary system
- Open tank may collect floatables

The HeadCell® has the advantage of not having any moving parts and not requiring construction of the deep circular tank required for the vortex system. A HeadCell® sized for the PIF would be 6 feet in diameter and have a total of 4 trays (10 feet deep overall).

#### 7.1.2.4 Cost Comparison

Table 7-1 summarizes estimates of equipment cost for the three types of grit removal systems. In general, equipment and installations costs for the three systems are very comparable.

<b>Table 7-1</b> <b>Comparison of Probable Cost for Grit Removal Technologies</b> <b>MCSD Wastewater Management Facility</b>						
	<b>Equipment</b>	<b>Grit Pumps</b>	<b>Classifier/ Cyclone</b>	<b>Concrete Tank</b>	<b>Electrical</b>	<b>Total</b>
Aerated Grit Chamber	\$174,000	Supplied	Supplied	\$35,000	\$52,200	\$261,200
Vortex- type Grit System	\$75,000	\$40,000	\$78,000	\$30,000	\$57,900	\$280,900
HeadCell®	\$90,000	\$40,000	\$78,000	\$25,000	\$42,900	\$275,900

## 7.2 Upgrade or Expand Existing Facultative System

The existing WWMF, a facultative lagoon system with supplemental aeration followed by treatment wetlands, is at capacity for organic loading and detention times are insufficient to provide required ammonia removal. Several alternatives for improving secondary treatment and nitrogen removal through an upgrade or expansion of the existing system were evaluated. These alternatives included:

- Increased Wetland Treatment
- Upgraded Aeration System (partial mix/complete mix)
- Use of a Nitrifying Filter

### 7.2.1 Increased Wetland Treatment

Wetland treatment has been shown to be an effective method of BOD removal when systems are loaded at rates less than 100 ppd/ac. The degree to which the systems are effective at removing nitrogen depends upon nitrification and denitrification reactions that are a function of detention time and temperature and upon the numbers of nitrifying organisms. Nitrifying organisms require oxygen and an adequate surface area on which to grow.

#### 7.2.1.1 Conversion of Ponds 2 and 3

It was noted in the performance review that most of the BOD and ammonia removal takes place in Ponds 1A and 1B, and Ponds 2 and 3 do not contribute significantly to BOD removal, especially in the summer. Conversion of Ponds 2 and 3 to treatment wetlands was considered as an alternative for improving secondary treatment capacity.

Based on earlier discussions of wetland capacity, it was determined that loading on treatment wetlands should not exceed 100 ppd/ac and optimally should be around 50 ppd/ac. The loadings presented in Table 7-2 indicate that if Ponds 2 and 3 were converted to wetlands and all four wetland cells are fed in series as wetland treatment cells, the loading would still exceed the recommended maximum of 100 ppd/ac at the design condition. If wetlands are to be used to improve secondary treatment, additional wetland area(s) will need to be constructed.

<b>Table 7-2</b> <b>Organic Loading on Wetlands Following Conversion of Ponds 2 and 3</b> <b>MCSO Wastewater Management Facility</b>							
	Projected Flow 2030 (MGD) <sup>1</sup>	Influent BOD <sup>2</sup> (mg/L) <sup>3</sup>	Removal	Pond 1A and 1B Effluent BOD		Area (acres)	Loading (ppd/ac) <sup>5</sup>
			(%)	(mg/L)	(ppd) <sup>4</sup>		
MMWWF <sup>6</sup>	2.13	244	65%	85	1,517	10.81	140
MMDWF <sup>7</sup>	1.60	272	80%	54	726	10.81	67
1. MGD: million gallons per day 2. BOD: Biochemical Oxygen Demand 3. mg/L: milligrams per Liter 4. ppd: pounds per day 5. ppd/ac: pounds per day per acre 6. MMWWF: Maximum Monthly Wet Weather Flow 7. MMDWF: Maximum Daily Wet Weather Flow							

### 7.2.1.2 Area Requirements

To determine the area of wetland treatment cells required for provision of required secondary treatment, kinetic coefficients developed from studies of existing Free Water Surface (FWS) wetlands were applied to design loadings from Stabilization Ponds 1A and 1B (Crites and Tchobanoglous, 1998).

#### A. Secondary Treatment (BOD Removal)

Secondary permit requirements of 30 mg/L BOD were assumed. The target effluent concentration was derived by subtracting 5 mg/L from the required effluent concentration, to account for BOD contributed by plant decay in the wetland cells, and then multiplying by a Coefficient of Reliability (COR) to account for variability in the wetlands effluent. A COR of 60% was used based on typical coefficients of variation for wetland treatment and assuming a statistical probability of 95% for meeting the criteria (Crites and Tchobanoglous, 1998).

Detention time for BOD removal:

$$t \text{ (days)} = -(\ln C/C_0)/k_w = 4.6 \text{ days}$$

Where:

$C_0$  = Influent = 87 mg/L BOD

$C_e$  = Target Effluent Quality = 30 mg/L BOD

$C_d$  = Added by Decay = 5 mg/L BOD

COR = Coefficient of Reliability (@ 95 % Reliability) = 0.60

$C = \text{COR}(C_e - C_d) = 15$

$t$  = detention time, days

$k_w$  = overall BOD removal-rate constant corrected for temperature, = 0.38 d<sup>-1</sup>

Area requirements can be calculated from the required detention time using the following formula:

$$\text{Area (acres)} = (Q \cdot t \cdot 3.07) / (dw \cdot n) = 19 \text{ Acres}$$

Where:

$Q_{\text{INF}}$  = Influent, MGD = 2.1 MGD (Projected MMWWF 2030)

$n$  = plant based void ratio = 0.65

$dw$  = depth of flow, ft = 2.3 ft

Conversion 3.07 ac.ft/MGD

Table 7-3 summarizes the design criteria for wetland treatment cells. Based on the preceding calculations, we estimated a total detention time of 4.6 days and an estimated area of 19 acres of wetland treatment would be required for BOD removal at projected design loadings.

<b>Table 7-3</b> <b>Design Criteria Free Water Surface Constructed Wetlands</b> <b>MCSO Wastewater Management Facility</b>	
<b>Secondary Treatment Wetlands</b>	
Maximum Areal Loading Rate (ARL)	<100 ppd <sup>1</sup> /acre
Design Areal Loading Rate (ARL)	50 ppd/ac
Hydraulic Retention Time (HRT)	3-5 days
Water Depth (Shallow)	0.2-1.5 feet
Water Depth (Deep)	>4 feet <sup>2</sup>
<b>Enhanced Treatment Wetlands</b>	
Hydraulic Retention Time (HRT)	10 days
<b>BOD<sup>3</sup> Removal Rate<sup>4</sup></b>	
$K_{\text{BOD}}$ (20 °C) <sup>5</sup>	0.68/day
$K_w$ (10.2 °C) <sup>6</sup>	0.38/day
$K_s$ (16.5 °C) <sup>7</sup>	0.55/day
<b>Ammonia Removal Rate<sup>8</sup></b>	
$K_{\text{NH}_3\text{-N}}$ (20 °C)	0.22/day
$K_w$ (10.2 °C)	0.14/day
$K_s$ (16.5 °C)	0.19/day
1. ppd: pounds per day 2. >: greater than 3. BOD: Biochemical Oxygen Demand 4. Temperature Correction for BOD removal theta = 1.06 5. $K_{\text{BOD}}$ : Removal rate for BOD 6. $K_w$ : Removal rate for winter 7. $K_s$ : Removal rate for summer 8. Temperature Correction for NH <sub>4</sub> -N removal theta = 1.048	

## B. Nutrient Removal

The rates of nitrification, the conversion of ammonia-nitrogen (NH<sub>3</sub>-N) to nitrate-nitrogen (NO<sub>3</sub>-N), and denitrification the subsequent conversion of NO<sub>3</sub>-N to nitrogen (N<sub>2</sub>) are much slower than those for BOD conversion, therefore the area of wetlands required for nitrogen removal will far exceed the area required for BOD and TSS removal. The expected effluent quality can be calculated based on published rates for nitrification and the actual NH<sub>3</sub>-N concentrations of Ponds 1A and 1B effluent.



In 2009, the average wet-weather concentration of the NH<sub>3</sub>-N leaving Ponds 1A and 1B was 29.5 mg/L and it is assumed an additional 9 mg/L of ammonia will be contributed by the organic nitrogen in Pond 1A and 1B effluent. At a detention time of 4.6 days in the treatment wetlands, the removal rate for nitrogen is 48% and the effluent concentration of ammonia is 20 mg/L.

The area required to provide an effluent with TKN concentration of 10 mg/L was calculated as follows:

$$t \text{ (days)} = -(\ln C/C_0)/k_w = 12 \text{ days}$$

Where:

C<sub>0</sub>= Influent = 38.5 mg/L NH<sub>3</sub>-N

C<sub>e</sub>= Target Effluent Quality = 7 mg/L NH<sub>3</sub>-N (TKN=10 mg/L)

t = detention time, days

k<sub>w</sub>= overall NH<sub>3</sub>-N removal-rate constant corrected for temperature, = 0.14 d<sup>-1</sup>

Area requirements can be calculated from the required detention time as follows:

$$\begin{aligned} \text{Area (acres)} &= (Q \cdot t \cdot 3.07) / (d_w \cdot n) \\ &= 52 \text{ Acres} \end{aligned}$$

Where:

Q<sub>INF</sub> = Influent , MGD = 2.1 MGD (Projected MMWWF 2030)

n = plant based void ratio = 0.65

d<sub>w</sub>= depth of flow, ft =2.3 ft

Conversion 3.07 ac.ft/MGD

### C. Reliability

The existing wetland treatment cells do not achieve expected rates of ammonia removal, and BOD removal is not reliable. Nitrification is limited by:

- Wetland cells are subject to high organic loads, which deplete oxygen and select heterotrophs (an organism that cannot fix carbon and uses organic carbon for growth) over nitrifiers (bacteria that grow by consuming inorganic nitrogen compounds).
- Wetland cells are not configured for optimal nitrogen removal. Deeper free water surface areas designed to optimize nitrification should precede the shallow cells planted with emergent vegetation, which is best at denitrification. It is difficult to maintain plants in open water surface areas due to plant predation. (The WWMF has experienced problems maintaining plants in the free water surface areas of the existing cells due to plant predation. A pilot test is being conducted using SAV in Ponds 3 and 4. The purpose of the pilot test is to improve performance of the enhancement wetlands. However, even if plants can be maintained in the open water areas due to improved plant selection, large areas are needed to reliably provide required rates of nitrification.)

### D. Feasibility and Cost

Within the existing plant site boundaries there are an estimated 24 acres that could be converted to wetlands. Land available for additional wetland treatment cells to the south and west of the

existing plant could provide the additional 8 acres required for secondary treatment; however, there is not enough area to provide the 52 acres required for reduction of total nitrogen levels to 10 mg/L. Prior to determining the availability of additional land, a preliminary opinion of probable cost (Table 7-4) was developed to determine feasibility using the following assumptions:

- Cells would be configured for plug flow with the ability to be bypassed.
- Gravity flow from one cell to the next would not be feasible therefore pump stations would be required to pump the discharge from each wetland treatment cell.
- It is assumed that all the wetlands treatment cells would need to be lined. Unless exempted, wastewater surface impoundments must be designed in accordance with Title 27 requirements for a Class II waste management units. The requirements include provisions for liners that meet a prescriptive standard or for an engineered alternative that provides equivalent protection. Engineered alternatives and/or exemptions to this requirement require further detailed analyses.

<b>Table 7-4</b> <b>Engineer's Opinion of Probable Cost for Wetland Treatment</b> <b>MCSD Wastewater Management Facility</b>				
Description	Unit	Unit Cost	Quantity	Total Cost
Mobilization 12%				\$1,845,235
<b>Convert Ponds 2 and 3 (5 Acres)</b>				
Lining	SF <sup>1</sup>	\$2.50	217,800	\$544,500
New Planting	SF	\$1.50	217,800	\$326,700
<b>Modify Ponds 4 and 5 (6 Acres)</b>				
Planting	SF	\$1.50	261,360	\$392,040
Lining	SF	\$2.50	261,360	\$653,400
<b>Additional Treatment Wetlands (8 Acres)</b>				
Excavation	CY <sup>2</sup>	\$20	38,720	\$774,400
Berm	CY	\$30	20,000	\$600,000
New Planting	SF	\$1.50	348,480	\$522,720
Lining	SF	\$2.50	348,480	\$871,200
Outlet structures	EA <sup>3</sup>	\$5,500	4	\$22,000
Pump Stations	EA	\$7,500	4	\$30,000
Collection Piping	LF <sup>4</sup>	\$100	800	\$80,000
<b>Enhancement Wetlands (32 acres)</b>				
Construction <sup>5</sup>	LS <sup>6</sup>	\$10,560,000	ALL	\$10,560,000
Construction Subtotal				\$17,222,195
Contingency 20%				\$3,444,439
Engineering 25%				\$4,305,548
Admin 4%				\$688,887
<b>Project Subtotal</b>				<b>\$25,661,070</b>
1. SF: Square Feet 2. CY: Cubic Yards 3. EA: Each 4. LF: Linear Feet 5. Cost of additional wetlands estimated at \$330,000/acre 6. LS: Lump Sum				

Advantages:

- Maintains use of existing stabilization ponds and natural treatment system

Disadvantages:

- The large areas required (coupled with pumping and lining costs) limit the feasibility of this alternative for secondary treatment and enhanced nutrient removal
- Preliminary estimates of project costs are more than four times the cost of alternatives providing equivalent treatment (as presented in subsequent sections)
- Has a lower reliability than mechanical systems

## 7.2.2 Aerated System/Nitrifying Filters

The existing facultative pond system with supplemental aeration does not have the capacity to treat projected organic loadings. However, higher loadings can be sustained if air is added in sufficient quantities to suspend the microbial population in the aerated basins. In aerated basins that are either partially or completely mixed, the kinetic constants governing BOD removal and nitrification are higher than those in a facultative system where the microbial population is not suspended.

In this section, increasing secondary treatment capacity by changing the existing facultative pond system to an aerated pond system was evaluated as a feasible alternative. The aerated pond system would be combined with a nitrifying filter bed to provide reliable nitrification of ammonia.

### 7.2.2.1 Partially Mixed System

In a partially mixed system, the oxygen requirements generally control the power input required. Air requirements based on projected BOD and TKN and NH<sub>3</sub>-N loadings and assuming a transfer efficiency of 2.0 pounds of Oxygen (O<sub>2</sub>) per horsepower-hour (hp-hr) were estimated to be 360 hp.

The reduction of BOD in a partial mix system was calculated based upon available detention time in the existing three pond system minus an allowance for construction of a nitrifying filter as follows:

$$\begin{aligned}C_n &= 1/[1+kt/n]^n C_0 \\&= (0.17)(C_0) \\&= 42 \text{ mg/L BOD}\end{aligned}$$

Where:

$k = 0.19 \text{ d}^{-1}$  (@ 10 °C)  
Detention time = 13 days (At Projected MMWWF 2030)  
 $C_n$  = Effluent BOD concentration in cell  $n$   
 $C_0$  = Influent BOD concentration = 244 mg/L  
 $n$  = number of cells in series = 3

### 7.2.2.2 High Performance Aerated Pond System

The High Performance Aerated Pond System (HPAS) also called the Dual Power Multicellular (DPMC) treatment process was developed as a modification to the conventional aerated lagoon

system and has been used with success in numerous installations in the U.S. The system is comprised of multiple cells aerated at different levels. The first cell consists of a reactor with a retention time of 1.5 to 2.5 days aerated to maintain complete mix conditions at a minimum of 30 horsepower per million gallons (hp/MG). The partially mixed cells following the suspended cell are aerated at 5 to 8 hp/MG—a level that permits the settleable solids to settle, but is sufficient to maintain aerobic conditions.

The deep sections of Ponds 1A and 1B are suitable for creation of complete mix cells using suspended baffles. The primary sections of these two ponds are as much as 13.5 feet deep—enough to use large aspirating aerators with integral blowers supplying fine bubble aeration and a high oxygen transfer efficiency.

The reduction of BOD in the combined system was calculated as follows

$$\begin{aligned}C_n &= 1/[1+kt/n]^n C_0 \\&= (0.26)(C_0) \\&= 63 \text{ mg/L BOD}\end{aligned}$$

Where:

Complete Mix:

$$\begin{aligned}k &= 1.7 \text{ d}^{-1} \text{ (@ } 10^\circ \text{C)} \\ \text{detention time} &= 2.25 \text{ days (At Projected MMWWF 2030)} \\ C_n &= \text{Effluent BOD concentration in cell } n \\ C_0 &= \text{Influent BOD concentration} = 244 \text{ mg/L} \\ n &= 1\end{aligned}$$

$$\begin{aligned}C_n &= 1/[1+kt/n]^n C_0 \\&= (0.31)(C_0) \\&= 20 \text{ mg/L BOD}\end{aligned}$$

Where:

Partial Mix:

$$\begin{aligned}k &= 0.19 \text{ d}^{-1} \text{ (@ } 10^\circ \text{C)} \\ \text{detention time} &= 8.35 \text{ days (@ MMWWF)} \\ C_n &= \text{Effluent BOD concentration in cell } n \\ C_0 &= \text{Influent BOD concentration} = 63 \text{ mg/L} \\ n &= 2\end{aligned}$$

### 7.2.2.3 Nitrifying Filter

Nitrifying filter beds are an innovative concept developed as a retrofit for constructed wetlands systems to meet ammonium-nitrogen (NH<sub>4</sub>-N) discharge requirements reliably (Reed et al., 2006). The rock media provide the substrate for a biofilm in much the same way as rocks would in a trickling filter or Recirculating Gravel Filter (RGF). However, filter beds are specifically designed to remove NH<sub>4</sub>-N following a secondary process to remove BOD and are subjected to much higher hydraulic loading rates than RGFs.

Design criteria for a filter bed area are based on attached growth processes and are related to the specific surface area of the media. Additional requirements include:

- Low BOD ( $\text{BOD}/\text{TKN} < 1$ )
- Aerobic conditions
- Surface moist at all times
- Sufficient alkalinity ( $8.6 \text{ mg alkalinity } (\text{CO}_3^{2-})/\text{mg NH}_3\text{-N oxidized}$ )

An equation relating ammonia loading to required surface area was developed based on curve fitting of performance data from other nitrification reactors. The equation for determining surface area was verified in a full scale application at Mandeville, Louisiana (Reed et al., 2006). In Mandeville, the filter bed followed a high performance aerated lagoon system similar in design to the combined aeration system described above.

Based on the equation cited above and assuming an effluent  $\text{NH}_3\text{-N}$  concentration of  $5 \text{ mg/L}$ , the required surface area is  $5,744$  square feet per pound of  $\text{NH}_3\text{-N}$  oxidized. If the filter is constructed using gravel with a surface area of  $57.8$  square feet per cubic foot ( $\text{SF}/\text{CF}$ ) and the maximum projected loading is  $525$  ppd  $\text{NH}_3\text{-N}$ , then approximately  $1,930$  cubic yards ( $\text{CY}$ ) of media will be required.

Nitrifying filters generally are 1 to 2 feet deep. Continuous feed is possible if aerobic conditions are maintained in the filter, for example through the use of recirculating pumps and a spray distribution system.

#### 7.2.2.4 Denitrification

The nitrifying filter is designed to convert ammonia-nitrogen ( $\text{NH}_3\text{-N}$ ) to nitrate-nitrogen ( $\text{NO}_3\text{-N}$ ) because nitrification is generally the rate limiting step in a free water surface wetland. Removal of the nitrate is accomplished through a combination of plant uptake and biological conversion or denitrification with bacterial conversion accounting for 90% of the removal.

The low DO environment in an established free water surface wetland is conducive to denitrification which generally can be accomplished at detention times of 2 to 4 days. As with other bacteriologically mediated processes, denitrification is affected by temperature and detention time and limiting conditions in the wetlands that occur during winter. Using a rate constant of  $0.247/\text{day}$  ( $1.0/\text{day}$  at  $20^\circ\text{C}$  corrected for a temperature of  $10^\circ\text{C}$ ), the required detention time to denitrify the effluent from the nitrifying filter is estimated to be 5 days.

Based on the projected MMWWF of  $2.1 \text{ MGD}$  and assuming a maximum depth of 3 feet in the wetlands, a detention time of 5 days equates to a required area of approximately 11 acres, which is the entire area of Ponds 2 and 3, and Wetland Cells 4 and 5. However, if the nitrifying filter is built so that flow can be recycled from the wetlands, the area required for denitrification can be reduced. Recycling also has the advantage of decreasing the soluble BOD concentration going into the filter.

### 7.2.2.5 Feasibility and Cost

Of the two aeration systems investigated, only the high performance aeration system could reliably achieve the BOD removal required to provide a BOD/TKN ratio of less than one prior to the nitrifying filter. In addition, the HPAS has a lower total power requirement and frees up area for additional wetlands to denitrify the filter bed effluent.

A preliminary layout for the combined aerated pond system and nitrifying filter bed is shown in Figure 7-1. An estimate of probable cost is presented in Table 7-5.

#### Advantages:

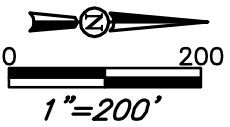
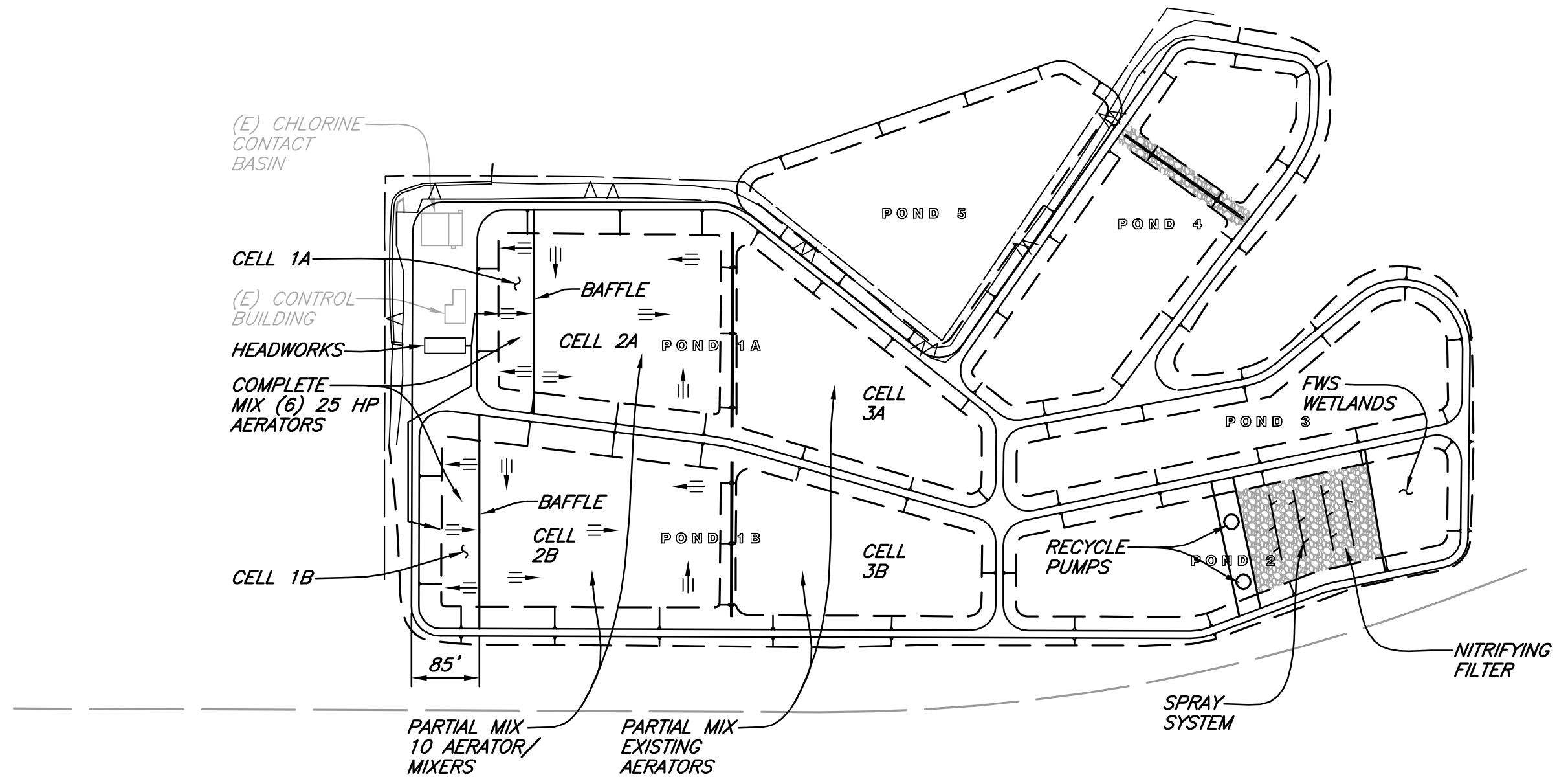
- Can reliably achieve BOD removal required
- Construction costs comparable to extended aeration, lower than other alternatives
- Maintains natural wetland treatment system
- Low biosolids production

#### Disadvantages:

- Cannot reliably provide effluent ammonia levels of less than 5 mg/L
- Large construction footprint
- More difficult to expand system; may require area outside existing footprint
- Higher power costs as compared to other alternatives
- When O&M costs including power are considered present value is higher than extended aeration
- Plugging of rock filter may require maintenance

<b>Table 7-5</b> <b>Engineer's Opinion of Probable Cost for High Performance Aeration System/Nitrifying Filter</b> <b>MCSD Wastewater Management Facility</b>				
Description	Unit	Unit Cost	Quantity	Total Cost
Mobilization 12%				\$522,757
Sludge Removal/Dewatering	LS <sup>1</sup>		ALL	\$200,000
Aerators 25 hp <sup>2</sup>	EA <sup>3</sup>	\$36,000	6	\$216,000
Aerators 5-7.5 hp	EA	\$15,000	10	\$150,000
Lining Aerated Basins	SF <sup>4</sup>	\$2.50	487,872	\$1,219,680
Lining Pond 2	SF	\$2.50	104,544	\$261,360
Lining Pond 3	SF	\$2.50	108,900	\$272,250
Plantings	SF	\$1.50	130,680	\$196,020
Baffles	LF <sup>5</sup>	\$75	1,500	\$112,500
<b>Nitrifying Filter</b>				
Filter Recycle Pumps	EA	\$15,000	2	\$30,000
Filter Structure	CY <sup>6</sup>	\$1,100	1,135	\$1,248,500
Filter Media	CY	\$35	2,000	\$70,000
Spray Distribution System	LS	\$30,000	ALL	\$30,000
Electrical	LS	\$175,000	ALL	\$175,000

I:\2008\008189-MCSD\008189-C, SAVED: 8/9/2011 3:14 PM NDOWNNEY, PLOTTED: 1/11/2012 3:06 PM, NATHAN DOWNEY



BASE MAP PROVIDED BY :  
WINZLER & KELLY, DATED JUNE 2005



McKinleyville Community Services District  
Wastewater Management Facility  
McKinleyville, CA

High Performance Aeration System (HPAS)/  
Nitrifying Filter (NF) Alternative 1  
SHN 008189

August 2011

008189-POND-ALT1

Figure 7-1



<b>Table 7-5</b> <b>Engineer's Opinion of Probable Cost for High Performance Aeration System/Nitrifying Filter</b> <b>MCSD Wastewater Management Facility</b>				
Description	Unit	Unit Cost	Quantity	Total Cost
Generator	EA	\$175,000	1	\$175,000
Construction Subtotal				\$4,879,067
Contingency 20%				\$975,813
Engineering 25%				\$1,219,767
Admin 4%				\$195,163
<b>Project Subtotal</b>				<b>\$7,269,810</b>
1. LS: Lump Sum	4. SF: square Feet			
2. hp: horsepower	5. LF: Linear Feet			
3. EA: Each	6. CY: cubic Yards			

### 7.3 Extended Aeration System Processes

In the performance analysis for natural treatment systems, the slow growth rate of nitrifiers leads to a requirement for long detention times in aerated pond systems and wetlands. To create a reliable process for nitrification, the lagoon process must be modified so that the solids age can be uncoupled from the Hydraulic Retention Time (HRT). This is accomplished through sedimentation in clarifiers with solids recycling which is the definition of an activated sludge or suspended growth process.

In the suspended growth process, the age or Solids Retention Time (SRT) of the bacterial population is managed and nitrifiers are selected for the maintenance of a high SRT. Two extended aeration processes were evaluated as appropriate for the WWMF:

- Suspended aeration chains and internal clarifiers installed in existing facultative ponds
- An oxidation ditch: an aeration basin constructed in an elliptically shaped channel followed by circular clarifiers

#### 7.3.1 Earthen Basin System

The in-basin extended aeration system uses suspended aerators and integral clarifiers sharing a common wall with the aeration basin, to convert facultative pond systems to extended aeration systems. In addition to providing fine bubble diffused air with high transfer efficiency, the suspended aerators mix large basin volumes efficiently. This has led to the development of systems that are designed with SRTs that are longer than most extended aeration systems.

As an alternative to the use of integral clarifiers, conventional stand-alone clarifiers can also be used with the in-basin extended aeration system.

##### 7.3.1.1 Effluent Quality

The in-basin extended aeration process results in reliable BOD removal, nitrification, and denitrification. The extended aeration system with solids recycle would provide the ability to

reliably nitrify and provide effluent  $\text{NH}_4$  (ammonium) at a concentration of less than 1 mg/L. In addition, the suspended aerators can be controlled to provide anoxic zones for denitrification reducing effluent nitrate concentrations to 3 to 4 mg/L. The system can reliably achieve TN concentrations, including an allowance for unmetabolized organic nitrogen, of 8 to 10 mg/L.

The long SRTs of 30 to 70 days provide process stability. Due to the large quantity of biological solids present, wide swings in organic and hydraulic loads can easily be handled without equipment or process adjustments. The excess biomass produced is well digested and stabilized.

### 7.3.1.2 General Configuration/Cost

Based on the projected loadings and flows developed in Section 3, several feasible configurations for an in-basin extended aeration system were developed. A configuration employing two basins placed end to end within a single earthen berm provided the most cost-effective use of the existing facultative pond. This configuration is shown in Figure 7-2. Aeration basin and clarifier dimensions were based on design criteria provided by two manufacturers of this type of system. Conventional stand-alone clarifiers would add an additional cost.

Table 7-6 presents a preliminary estimate of probable cost for an extended aeration system constructed within MCSD WWMF Pond 1B.

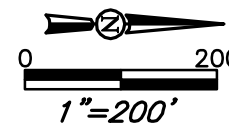
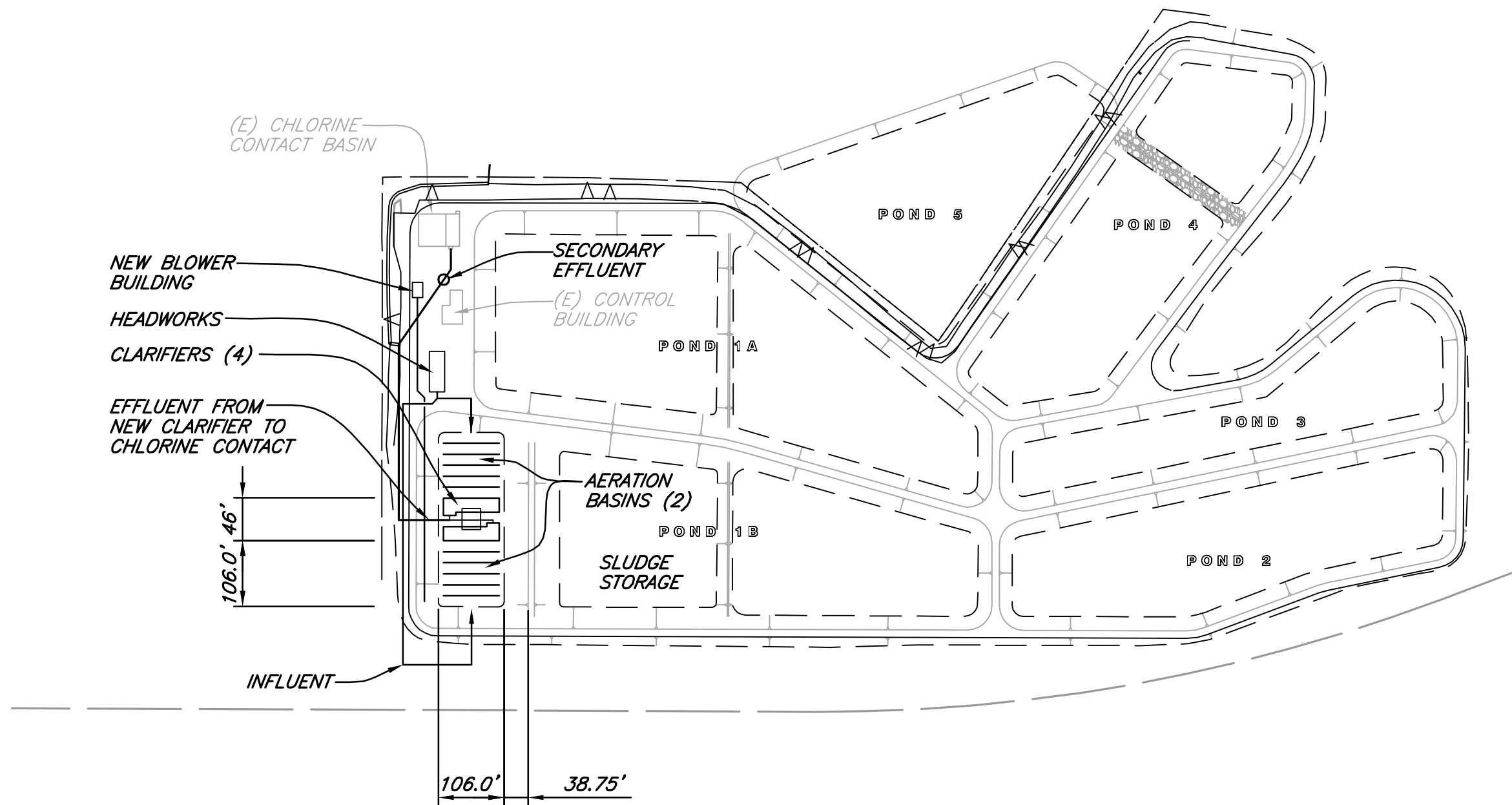
#### Advantages:

- High effluent quality with reliably low ammonia, nitrate and total nitrogen
- Efficient mixing resulting in reduced aeration requirements and power cost
- Long sludge age (greater than 60 days) provides very stable process tolerant to shock loads
- Low volume highly stabilized solids
- In-basin construction results in lower cost than other extended aeration systems (such as oxidation ditch)

#### Disadvantages:

- Aeration basin difficult to take off line (although not usually necessary for the range of flows at MCSD WWMF)
- Integral clarifiers may have issues with RAS control which can be addressed through modifications to the RAS pumps or through the use of conventional clarifiers

I:\2008\008189-MCSD\008189-C, SAVED: 8/9/2011 3:13 PM NDOWNNEY, PLOTTED: 1/11/2012 3:08 PM, NATHAN DOWNEY



BASE MAP PROVIDED BY :  
WINZLER & KELLY, DATED JUNE 2005



McKinleyville Community Services District  
Wastewater Management Facility  
McKinleyville, CA

In-Basin Extended Aeration System  
Alternative 2  
SHN 008189

August 2011

008189-POND-ALT2

Figure 7-2

<b>Table 7-6</b> <b>Engineer's Opinion of Probable Cost for Suspended Aerators and Integral Clarifiers</b> <b>MCSD Wastewater Management Facility</b>				
Description	Unit	Unit Cost	Quantity	Total Cost
Mobilization 12%				\$534,004
<b>Earthwork</b>				
Sludge Removal	LS <sup>1</sup>	\$300,000	ALL	\$300,000
Excavation	CY <sup>2</sup>	\$30	300	\$9,000
Foundation stabilization	CY	\$35	150	\$5,250
Fill (berm)	CY	\$32	12,500	\$400,000
Aeration Basin Lining	SF <sup>3</sup>	\$2.50	52,000	\$130,000
Sludge Pond Lining	SF	\$2.50	140,000	\$350,000
<b>Structural</b>				
Concrete Walls	CY	\$1,200	200	\$240,000
Inclined Base	CY	\$1,400	160	\$224,000
Blower Building	SF	\$250	600	\$150,000
<b>Equipment</b>				
Equipment	LS	\$1,368,000	ALL	\$1,368,000
Biolac Installation	LS	\$410,400	ALL	\$410,400
RAS Pumps <sup>4</sup>	EA <sup>5</sup>	\$6,000	4	\$24,000
Electrical	LS	\$356,880	ALL	\$356,880
Generator	EA	\$175,000	1	\$175,000
<b>Mechanical</b>				
Grating	SF	\$50	600	\$30,000
Railing	LF <sup>6</sup>	\$75	350	\$26,250
Influent 12-inch	LF	\$160	550	\$88,000
Effluent 14-inch	LF	\$200	450	\$90,000
RAS 10-inch	LF	\$100	300	\$30,000
WAS <sup>7</sup> 4-inch	LF	\$65	50	\$3,250
Air manifold 10-inch	LF	\$100	200	\$20,000
Additional Yard Piping	LS	\$20,000	ALL	\$20,000
Construction Subtotal				\$4,984,034
Contingency 20%				\$996,807
Engineering 25%				\$1,246,008
Admin 4%				\$199,361
<b>Project Subtotal</b>				<b>\$7,426,210</b>
1. LS: Lump Sum 2. CY: Cubic Yards 3. SF: Square Feet 4. RAS: Return Activated Sludge; assume installation of modified air lift or Geyser Pumps 5. EA: Each 6. LF: Linear Feet 7. WAS: Waste Activated Sludge				

### 7.3.2 Oxidation Ditch

An oxidation ditch is another extended aeration system that uses long SRTs for biological oxidation and nitrification. Typical oxidation ditch treatment systems consist of a single or multi-channels within a ring, oval or horse-shoe configuration. Horizontally or vertically mounted aerators provide circulation, oxygen transfer, and aeration in the ditch.

Oxidation ditches are applicable wherever activated sludge extended aeration systems are appropriate. Oxidation ditch systems have a larger footprint than conventional treatment systems, but can be less expensive to construct and operate due to the efficiency of the aeration and mixing. The oxidation ditch systems are generally followed by stand-alone clarifiers.

### 7.3.2.1 Effluent Quality

The effluent quality produced by oxidation ditch systems is similar to other extended aeration systems with concentrations: BOD 10 mg/L, TSS 15 mg/L, and NH<sub>3</sub> (ammonia) 1 mg/L. The systems can be configured to denitrify either by addition of a stand-alone anoxic basin with recycling or with an anoxic internal ring.

### 7.3.2.2 General Configuration/Cost

The two-train oxidation ditch system presented in Figure 7-3 was sized based on projected flows and loadings. The aeration basins shown provide an HRT of 20 hours at the projected MMWWF.

A budgetary estimate of aeration equipment costs was obtained from a manufacturer of oxidation ditch systems. The aeration system design was based on oxygen requirements to treat projected maximum month BOD and NH<sub>3</sub> loading. Based on a transfer efficiency of 3 pounds of Oxygen per Brake Horsepower (O<sub>2</sub>/BHP) the total power requirement for oxidation of BOD and NH<sub>4</sub>-N was 98 hp. Aeration is provided by four, 25-hp combination aerator/mixers, each with a 5-hp regenerative blower. The system is completely mixed with a mixing intensity of 113 hp/MG.

At a design SRT of 18 days, the oxidation ditch system is designed to nitrify completely. Two anoxic denitrification basins were included in the layout for the proposed alternative and in the opinion of probable cost (Table 7-7). Also included are two, 50-foot diameter circular clarifiers designed to provide a surface overflow rate of 800 gallons per day per square foot (gpd/SF) at the projected peak day flow of 3.08 MGD.

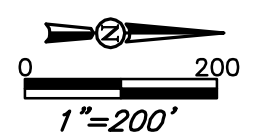
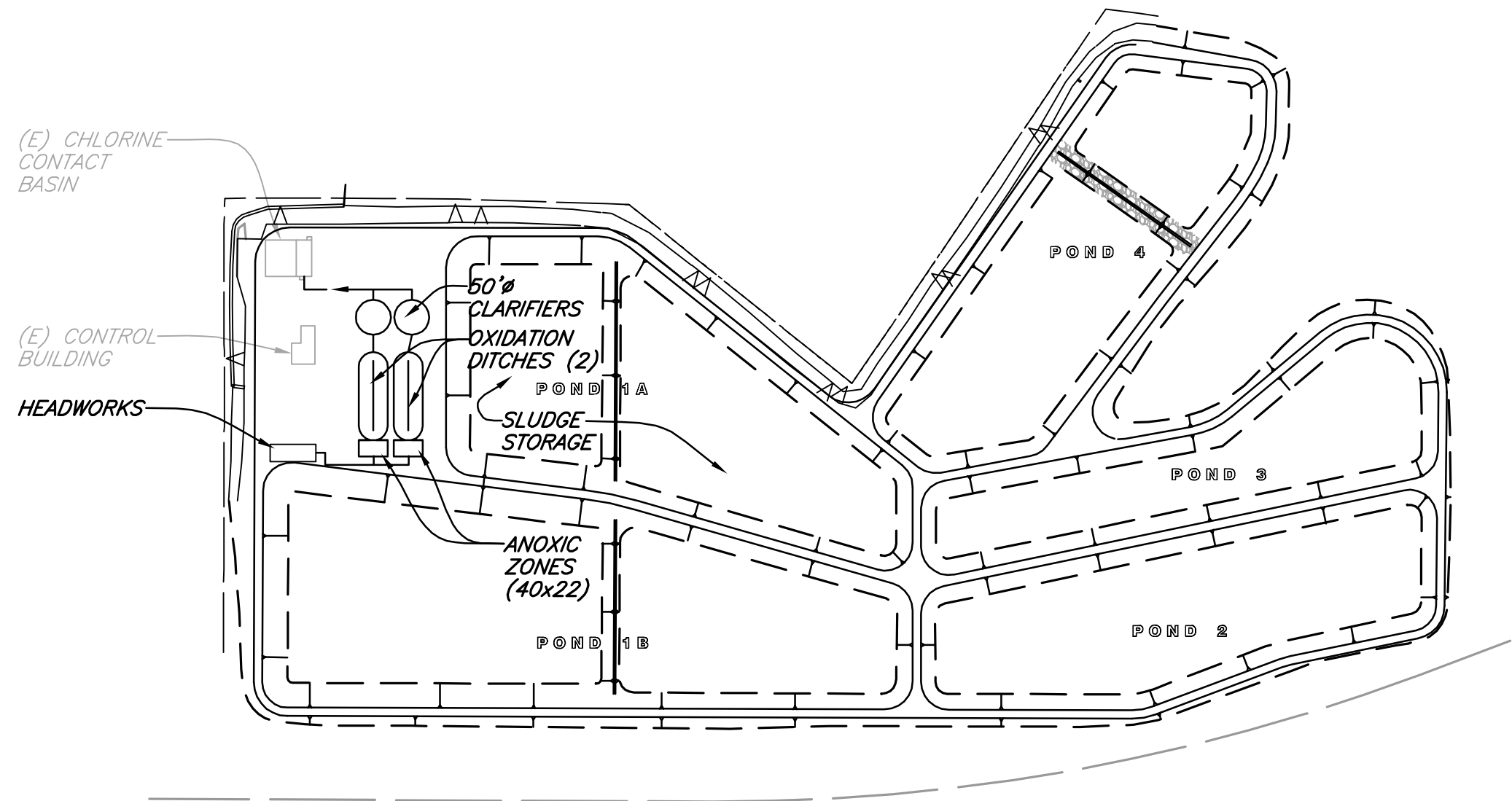
#### Advantages:

- Long sludge age (greater than 30 days) provides stable process tolerant to shock loads
- Stabilized solids intermediate in volume (between in-basin extended aeration and MBR)
- Can use aerators/mixers combination so aerators can be turned off for denitrification
- Intermediate in energy efficiency, power use

#### Disadvantages:

- Large footprint and concrete construction results in high costs
- Need for concrete clarifiers adds to cost and footprint

I:\2008\008189-MCSD\008189-C, SAVED: 10/3/2011 5:07 PM, NDOWNNEY, PLOTTED: 10/4/2011 8:52 AM, NATHAN DOWNEY



BASE MAP PROVIDED BY :  
WINZLER & KELLY, DATED JUNE 2005

 Consulting Engineers & Geologists, Inc.	McKinleyville Community Services District Wastewater Management Facility McKinleyville, CA		Extended Aeration Oxidation Ditch Alternative 3 SHN 008189	
	October 2011	008189-POND-ALT3	Figure 7-3	



<b>Table 7-7</b> <b>Engineer's Opinion of Probable Cost for Oxidation Ditch/Circular Clarifiers</b> <b>MCSD Wastewater Management Facility</b>				
Description	Unit	Unit Cost	Quantity	Total Cost
Mobilization 12%				\$641,808
<b>Earthwork</b>				
Sludge Removal/Dewatering	LS <sup>1</sup>	\$300,000	ALL	\$300,000
Fill (berm)	CY <sup>2</sup>	\$32	12,000	\$384,000
Additional Fill	CY	\$30	22,500	\$675,000
Line Sludge Pond	SF <sup>3</sup>	\$2.50	130000	\$325,000,
<b>Structural</b>				
Concrete Wall	CY	\$1,400	400	\$560,000
Oxidation Ditch Floor	CY	\$1,200	400	\$480,000
Anoxic Basins FS	CY	\$1,200	65	\$78,000
Anoxic Basins Wall	CY	\$1,200	100	\$120,000
Clarifiers Suspended	CY	\$1,400	200	\$280,000
Clarifiers Slab	CY	\$1,200	145	\$174,000
Clarifier Distribution Box	LS	\$60,000	ALL	\$60,000
<b>Equipment</b>				
Aeration Equipment	LS	\$504,000		\$504,000
Mixer	EA <sup>4</sup>	\$9,000	2	\$18,000
Clarifier Drives	LS	\$200,000	ALL	\$200,000
Launderer Weirs	LS	\$15,000	ALL	\$15,000
WAS <sup>5</sup> Pumps	EA	\$7,500	3	\$22,500
RAS <sup>6</sup> Pumps	EA	\$22,000	3	\$66,000
Scum Pumps	EA	\$4,500	2	\$9,000
Electrical	LS	\$300,000	ALL	\$300,000
Generator	EA	\$175,000	1	\$175,000
Installation	LS	\$228,600	ALL	\$228,600
<b>Mechanical</b>				
Catwalks	SF	\$100	350	\$35,000
Grating	SF	\$50	1,000	\$50,000
Railing	LF <sup>7</sup>	\$75	900	\$67,500
Influent 12-inch	LF	\$160	550	\$88,000
Effluent 14-inch	LF	\$200	450	\$90,000
RAS Piping 6-inch	LF	\$85	280	\$23,800
Additional Yard Piping	LS	\$20,000	ALL	\$20,000
Construction Subtotal				\$5,990,208
Contingency 20%				\$1,198,042
Engineering 25%				\$1,497,552
Admin 4%				\$239,608
<b>Project Subtotal</b>				<b>\$8,685,802</b>
1. LS: Lump Sum 2. CY: Cubic Yards 3. SF: Square Feet 4. EA: Each 5. WAS: Waste Activated Sludge 6. RAS: Return Activated Sludge 7. LF: Linear Feet				

## 7.4 Conventional Activated Sludge with Biological Nutrient Removal

Conventional activated sludge systems can be modified to provide reliable biological nutrient removal. Biological Nutrient Removal (BNR) is defined as the process by which concentrations of nitrogen and/or phosphorous in plant effluent are reduced to levels below that which would be attainable through secondary treatment only. All of the treatment alternatives evaluated include BNR for nitrogen removal, the removal of nitrogen through nitrification of ammonia and subsequent denitrification of nitrate and nitrite. Alternatives have been evaluated in order of increasing complexity. Conventional activated sludge systems configured for nitrification and denitrification are more mechanically complex than Alternatives 1 through 3.

Activated sludge systems are the most extensively used secondary treatment systems nationally; and numerous configurations have been developed for BNR. The Modified Ludzack-Ettinger (MLE) Process, an activated sludge system with an initial anoxic stage followed by an aerobic stage has a proven track record for total nitrogen (TN) removal and will be used as the basis of treatment for Alternative Number 4.

### 7.4.1 Process Description

Nitrification occurs in the aeration basin usually in the second half of a plug flow configuration where BOD/TKN ratio is reduced because of oxidation of BOD. To promote denitrification of nitrates and the subsequent removal of nitrogen from the system as gas, the effluent from the aeration basin is returned to an anoxic zone. The process of denitrification adds alkalinity and oxygen back to the system, replenishing some of what has been removed by nitrification in the aeration basin.

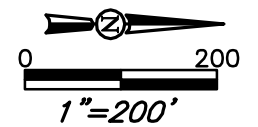
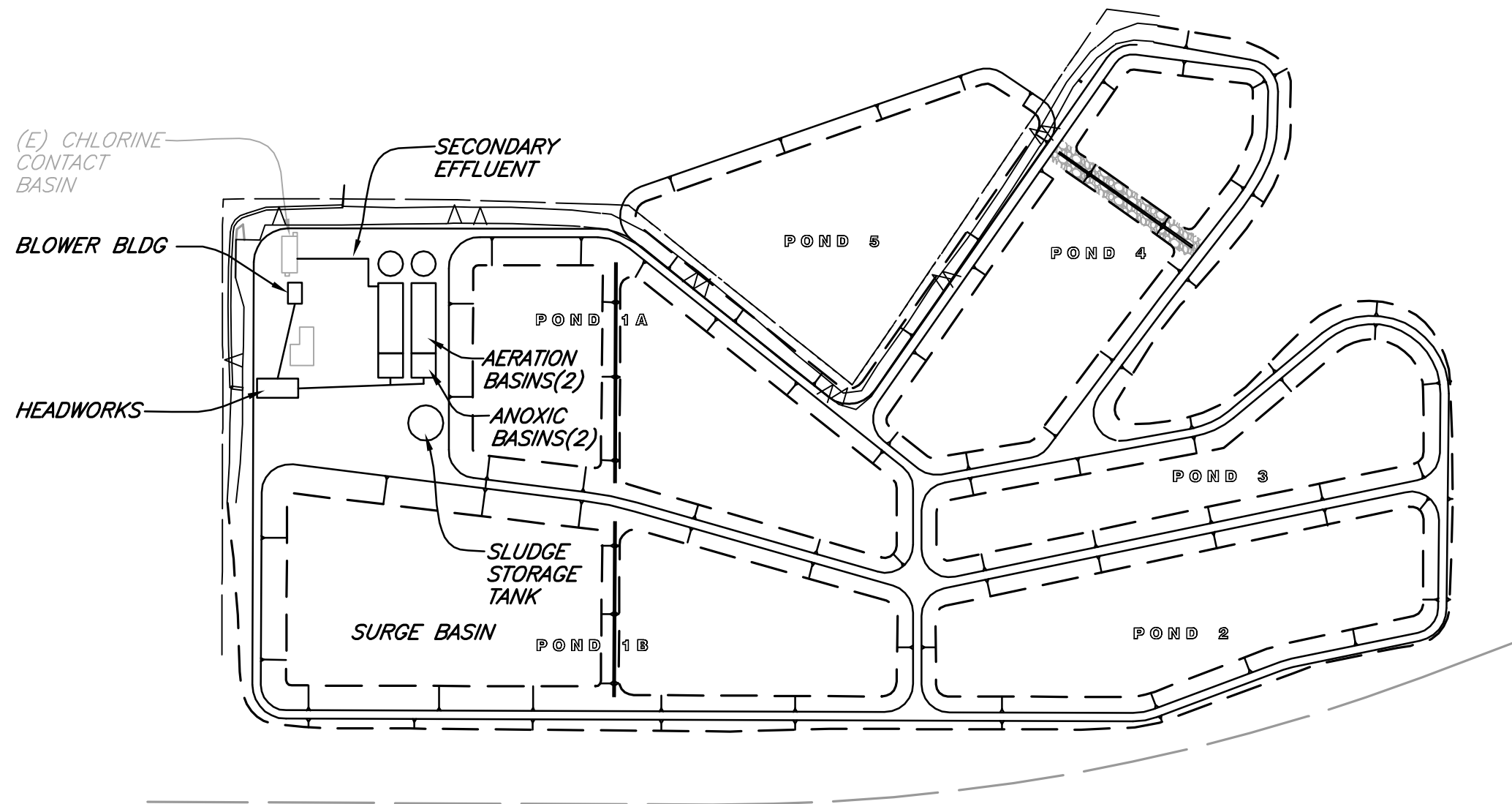
The pumps that recycle mixed liquor from the aeration basin back to the anoxic zone are called Mixed Liquor Recycle (MLRS) pumps. These are large solids handling pumps with the capability of pumping two to four times the influent flow. The anoxic basin, which must be provided with mixing, typically is equipped with submersible mixers.

### 7.4.2 Configuration and Cost

A layout for an activated sludge system designed to remove nitrogen is presented in Figure 7-4. Costs are presented in Table 7-8. The layout is based on the assumption that the system will need to treat the projected MMWWF of 2.137 MGD and that flows greater than this will be stored in the surge basin (approximately 19 MG). The aeration basins shown provide retention times of 6-12 hours at MMWWF; the anoxic zones approximately 2 hours. Two, 50-foot diameter clarifiers are similar to those required for Alternative 3, based on parallel oxidation ditches.

Cost does not include sludge dewatering or long-term storage. Sludge production exceeds that which could feasibly be stored in a sludge lagoon for periodic removal especially if Pond 1B is lined to provide flow equalization. Biosolids management costs are included in the evaluation of Operations and Maintenance (O&M) costs for the various treatment options in Section 7.5, based on the assumption that solids will be hauled to another facility for dewatering and disposal. A storage tank has been provided in the construction cost.

I:\2008\008189-MCSD\008189-C, SAVED: 10/4/2011 8:50 AM NDOWNEY, PLOTTED: 10/4/2011 8:51 AM, NATHAN DOWNEY



BASE MAP PROVIDED BY :  
WINZLER & KELLY, DATED JUNE 2005



McKinleyville Community Services District  
Wastewater Management Facility  
McKinleyville, CA

Conventional Activated Sludge (MLE)  
Alternative 4  
SHN 008189

October 2011

008189-POND-ALT4

Figure 7-4

<b>Table 7- 8</b> <b>Engineer's Opinion of Probable Cost for Activated Sludge With BNR</b> <b>MCSD Wastewater Management Facility</b>				
Mobilization	LS <sup>1</sup>	\$810,936	ALL	\$810,936
<b>Earthwork</b>				
Sludge Removal /Dewatering	LS	\$300,000	ALL	\$300,000
Fill (berm)	CY <sup>2</sup>	\$32	12,000	\$378,000
Additional Fill	LS	\$30	22,500	\$675,000
Surge basin	SF <sup>3</sup>	\$3	240,000	\$600,000
<b>Structural</b>				
Sludge Storage Tank	LS	\$175,000	ALL	\$175,000
Bower Bld.	LS	\$150,000	ALL	\$150,000
Tank Walls	CY	\$1,400	550	\$770,000
Slab	CY	\$1,200	800	\$960,000
PS (Dry Pit)	LS	\$150,000	ALL	\$150,000
Clarifiers	CY	\$1,400	200	\$280,000
Clarifiers	CY	\$1,200	145	\$174,000
Clarifier Distribution Box	LS	\$60,000	ALL	\$60,000
<b>Equipment</b>				
Aeration Equipment	LS	\$300,000	ALL	\$300,000
Diffusers	LS	\$240,000	ALL	\$240,000
Mixers	EA <sup>4</sup>	\$12,000	2	\$24,000
Clarifier Drives	LS	\$200,000	ALL	\$200,000
Launderer Weirs	LS	\$15,000	ALL	\$15,000
WAS <sup>5</sup> Pumps	EA	\$7,500	3	\$22,500
RAS <sup>6</sup> Pumps	EA	\$22,000	3	\$66,000
MLRS <sup>7</sup>	EA	\$30,000	3	\$90,000
Scum Pumps	EA	\$4,500	2	\$9,000
Generator	LS	\$175,000	ALL	\$175,000
Installation	LS	\$301,800	ALL	\$301,800
Electrical	LS	\$350,000	ALL	\$350,000
<b>Mechanical</b>				
Catwalks	SF	\$100	200	\$20,000
Railing	LF <sup>8</sup>	\$75	260	\$19,500
Recirculation	LF	\$120	260	\$31,200
Influent 12-	LF	\$160	550	\$88,000
Effluent 14-inch	LF	\$200	450	\$90,000
RAS Piping 6-inch	LF	\$85	280	\$23,800
Additional Yard Piping	LS	\$20,000	ALL	\$20,000
Construction Subtotal				\$7,568,736
Contingency 20%				\$1,513,747
Engineering 25 %				\$1,892,184
Admin 4%				\$302,749
<b>Project Subtotal</b>				<b>\$10,974,667</b>
1. LS: Lump Sum 2. CY: Cubic yards 3. SF: Square Foot 4. WAS: Waste Activated Sludge 5. RAS: Return Activated Sludge 6. MLRS: Mixed Liquor Recycle (pumps) 7. LF: Linear Foot				

Advantage:

- Reliability and proven track record

Disadvantages:

- High capital cost
- Higher annual power costs than suspended aeration due to MLRS pumps and mixers
- Higher sludge production than extended aeration systems
- Increased mechanical complexity
- Not as forgiving of shock loads as in-basin extended aeration with large sludge ages

## 7.5 Membrane Bioreactors

Membrane Bioreactors (MBRs) combine membrane technology with the activated sludge process to provide secondary and tertiary treatment in the same reactor vessel. With this treatment technology, microfiltration modules replace the clarification step with a membrane sheet providing liquid-solid separation. Suspended solids can be removed completely producing very high quality (almost bacteria-free) treated water.

### 7.5.1 Process Description

Membrane bioreactors use a hollow fiber, ultra filtration membrane immersed within an activated sludge tank with very high mixed liquor. With a pore size less than 0.1  $\mu\text{m}$ , the membrane is a complete physical barrier to the mixed liquor solids, bacteria, and most viruses. A vacuum varying between 2 and 9 pounds per square inch (psi) is applied to a head connecting the membrane modules through the use of a centrifugal pump. The treated water is drawn through the hollow fibers and pumped out as high quality effluent. Air in the aeration basin scours the membrane and keeps it from fouling.

Liquid is periodically pumped back through the membrane in a pulse, which coupled with a membrane air scour system, cleans the membrane by forcing solids away from it. Other components of the treatment system include pumps for inducing the vacuum, mixed liquor recycle pumps, membrane air scour blowers, chemical feed system for membrane cleaning, and an aeration system. Many of the treatment components are similar to those contained in an activated sludge system, although there is no need for a return activated sludge pumping system because there is no clarifier. MLRS pumps keep the reactor solids mixed, and solids are wasted from the recycle stream.

The MBR systems investigated employ ultra fiber hollow tube membranes, following biological treatment in high mixed liquor aeration systems. The filtered wastewater, or permeate, is pulled through the membrane by a vacuum pumping system. The biological treatment preceding the membranes generally have an SRT of 10 to 15 days, sufficient for nitrification but not as long as the extended aeration systems discussed in the previous sections. The aeration basin(s) are designed to provide a hydraulic retention time of 6 to 8 days.

With a membrane bioreactor the need for a secondary clarifier is eliminated because aeration and clarification can be carried out in a single reactor with the membranes immersed in the aeration basin or alternatively in a separate module following aeration. Performance is considered highly reliable because of the physical barrier provided by the membrane, which in the case of membrane bioreactors is an “ultra-filtration” membrane.

## 7.5.2 Configuration and Cost

MBRs are essentially a clarifier and filter in an activated sludge process containing very high mixed liquor concentrations. Because of the high sludge concentration and reactor capacity, the aeration volume is significantly reduced, allowing the entire process to have a small footprint.

Two budgetary proposals for membrane equipment were obtained for the purpose of providing an Engineer’s opinion of probable cost. The MemPulse™ system manufactured by Siemens Inc. was paired with an oxidation ditch configuration with mechanical aerators, and the Zenon™ system manufactured by General Electric followed conventional activated sludge basins provided with anoxic cells for denitrification. The proposed layout of the membrane system is presented in Figure 7-5. The costs presented in Table 7-9 are based on the Zenon™ system, but both estimates were similar, resulting in estimates of project cost close to \$13 million.

The proposals and layout are based on treating the projected MMWWF of 2.137 MGD and storing approximately 19 MGD in a surge basin. The amount stored is equal to the sum of the projected peak week for six days, PDAF for 20 hours, and PIF for four hours.

Membrane treatment systems provide very high quality effluent but are expensive systems to construct and operate with equipment cost and construction cost more than double the cost of other secondary systems evaluated. One advantage that a membrane treatment system provides includes a small footprint, which is not a significant benefit for MCSD given the amount of land available near the existing facility. Another advantage is the high quality effluent. In comparison to suspended air systems, the membrane treatment system will provide greater reliability for suspended solids removal and slightly greater BOD removal. The high effluent quality also allows disinfection requirements to be more reliably met by reducing interference with the chlorine system.

### Advantages:

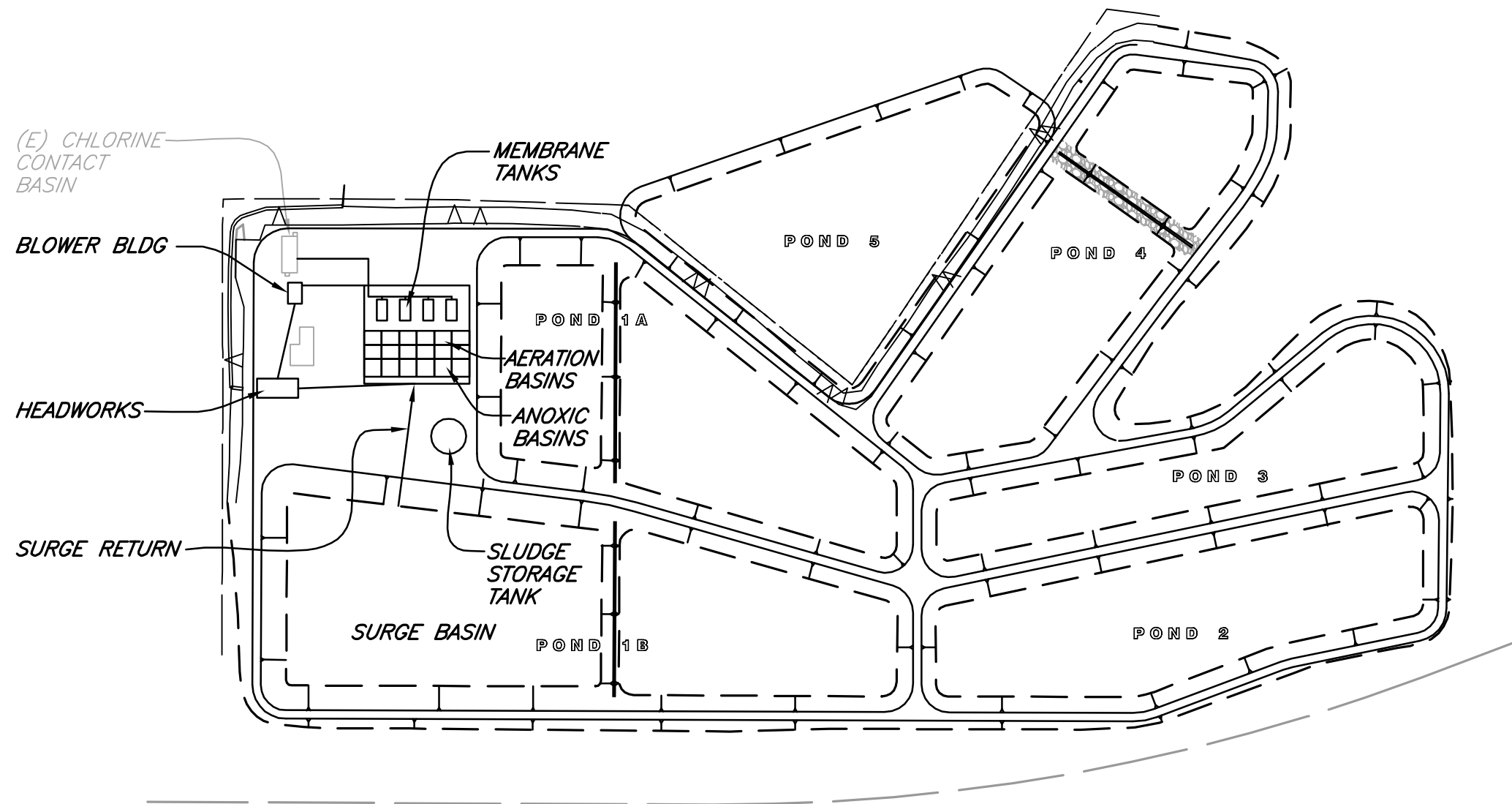
- Consistently high effluent quality
- Small footprint
- Reduced disinfection costs
- Can provide effluent suitable for municipal reuse without additional treatment
- Ability to meet more stringent discharge requirements than currently permitted

### Disadvantages:

- Higher operational complexity than extended air process
- High capital cost
- Additional mechanical and instrumentation maintenance required
- Higher sludge production than extended air processes



I:\2008\008189-MCSD\008189-C, SAVED: 10/4/2011 8:52 AM NDOWNEY, PLOTTED: 10/4/2011 8:52 AM, NATHAN DOWNEY



BASE MAP PROVIDED BY :  
WINZLER & KELLY, DATED JUNE 2005



McKinleyville Community Services District  
Wastewater Management Facility  
McKinleyville, CA

Membrane Bioreactor (MBR)  
Alternative 5  
SHN 008189

October 2011

008189-POND-ALT5

Figure 7-5

<b>Table 7-9</b> <b>Engineer's Opinion of Probable Cost for Membrane Treatment System</b> <b>MCSD Wastewater Management Facility</b>				
Description	Unit	Unit Cost	Quantity	Total Cost
Mobilization 12%				\$1,017,876
<b>Earthwork</b>				
Sludge Removal/Dewatering	LS <sup>1</sup>	\$300,000	ALL	\$300,000
Fill (Berm)	CY <sup>2</sup>	\$32	12,000	\$384,000
Surge Basin	SF <sup>3</sup>	\$2.50	240,000	\$600,000
<b>Structural</b>				
Slab	CY	\$1,200	800	\$960,000
Aeration Basin	CY	\$1,400	800	\$1,120,000
Sludge Storage Tank	LS	\$175,000	All	\$175,000
Blower Building	SF	\$250	600	\$150,000
<b>Equipment</b>				
Blower Equipment	LS	\$432,000	ALL	\$432,000
Membranes	LS	\$2,400,000	ALL	\$2,400,000
WAS <sup>4</sup> Pumps	EA <sup>5</sup>	\$7,500	3	\$22,500
RAS <sup>6</sup> Pumps	EA	\$22,000	3	\$66,000
Scum Pumps	EA	\$4,500	2	\$9,000
Installation	LS	\$859,800	ALL	\$859,800
Electrical	LS	\$550,000	ALL	\$550,000
Generator	EA	\$175,000	1	\$175,000
<b>Mechanical</b>				
Sidewalks	SF	\$50	1750	\$87,500
Railing	LF <sup>7</sup>	\$75	1,200	\$90,000
Influent 12-inch	LF	\$160	100	\$16,000
Effluent 14-inch	LF	\$200	200	\$40,000
RAS Piping 6-inch	LF	\$85	300	\$25,500
Additional Yard Piping	LS	\$20,000	ALL	\$20,000
Construction Subtotal				\$9,500,176
Contingency 20%				\$1,900,035
Engineering 25%				\$2,375,044
Admin 4%				\$380,007
<b>Project Subtotal</b>				<b>\$14,155,262</b>
1. LS: Lump Sum 2. CY: Cubic Yards 3. SF: Square Feet 4. WAS: Waste Activated Sludge 5. EA: Each 6. RAS: Return Activated Sludge 7. LF: Linear Feet				

## 7.6 Comparison of Secondary Treatment Options

This section provides a detailed comparison of secondary treatment alternatives and a recommendation regarding the preferred alternative as the project moves forward. Of the secondary treatment alternatives that were evaluated, there are five alternatives that would provide the required treatment for projected flows and loadings at a feasible cost:

- HPAS followed by nitrifying filter and wetlands
- In-basin extended aeration system with suspended aerators and integral clarifiers
- Oxidation ditch
- Activated Sludge with BNR (MLE process)
- Membrane Bioreactor (MBR)

Alternatives that were considered, but were determined to be infeasible due to high capital cost, land requirements, or other physical constraints, include expanded treatment and enhancement wetlands. Treatment and enhancement wetlands were discussed in the previous section and the probable cost of the wetlands construction (\$24 M) was determined a limiting factor.

The five alternatives considered viable were evaluated based on the following evaluation criteria:

- Effectiveness
- Cost
- Implementability
- Public Acceptance
- Regulatory Issues

### 7.6.1 Effluent Quality

Secondary treatment alternatives were evaluated based on the ability to provide an effluent that complies with current NPDES permit requirements for BOD, TSS, and  $\text{NO}_3\text{-N}$  concentrations, and to provide  $\text{NH}_3\text{-N}$  removal to levels that ensure compliance with whole effluent toxicity requirements when the facility is discharging to the Mad River. Nitrogen removal is an important indicator of current and anticipated effectiveness because in addition to anticipated permit requirements for  $\text{NH}_3\text{-N}$ , the concentration of TN is an important component of the area required for land application.

As Table 7-10 shows, all of the alternatives investigated provide significant nitrogen removal and ammonia reduction, but Alternative 1 cannot reliably provide effluent ammonia levels of less than 5 mg/L and, therefore, may not meet anticipated permit limits. Based on *EPA National Recommended Water Quality Criteria to Protect Freshwater Aquatic Life* (EPA, 2003), it is expected that new ammonia limits for discharge to the Mad River will be less than 5 mg/L and may be as low as 1 mg/L.

<b>Table 7-10</b> <b>Anticipated Average Monthly Effluent Quality</b> <b>MCSD Wastewater Management Facility</b> <b>(mg/L)<sup>1</sup></b>					
	<b>BOD<sup>2</sup></b>	<b>TSS<sup>3</sup></b>	<b>NH<sub>3</sub>-N<sup>4</sup></b>	<b>NO<sub>3</sub>-N<sup>5</sup></b>	<b>TN<sup>6</sup></b>
Aerated Pond System/Nitrifying Filter	20	20	5	5-10	10-15
In-basin Extended Aeration	10	10	<1 <sup>7</sup>	4	8
Oxidation Ditch	10	15	1	4	10
Activated Sludge with BNR (MLE Process)	10	10	1	4	8
Membrane Treatment	3	1	1	4	5 <sup>8</sup>
Current Mad River Discharge Limits	45	83	NA <sup>9</sup>	10	NA
Current Land Reclamation Limits	45	83	NA	10	NA
Anticipated Mad River Discharge Limits	30	30	1-5	10	10
Anticipated Land Reclamation Limits	30	30	NA	10	10
Anticipated Land Disposal Limits	30	30	NA	10	10
1. mg/L: milligrams per Liter 2. BOD: Biochemical Oxygen Demand 3. TSS: Total Suspended Solids 4. NH <sub>3</sub> -N: Ammonia-Nitrogen 5. NO <sub>3</sub> -N: Nitrate-Nitrogen 6. TN: Total Nitrogen 7. <: less than 8. Tertiary treatment and or chemical addition required to get below 5 mg/L 9. NA: Not Applicable					

The extended aeration systems provide a higher quality effluent, and are more reliable than the improved lagoon system. The extended aeration systems have proved successful for removal of BOD and nutrients and performance guaranties can be obtained from system manufacturers. MBRs also produce high quality effluent, but at substantial additional cost.

Extended aeration systems, MBRs and MLE systems will reliably meet anticipated discharge standards for NH<sub>4</sub>-N. MBR effluent is very low in TSS and can meet Class 2A reuse standards for municipal reuse without additional treatment. This gives them an advantage over extended aeration and MLE systems that would require a tertiary sand filter to provide the low turbidity water required to meet this purple pipe standard.

The ability of membrane bioreactors to provide higher quality water is reflected in a higher score for reliability and effectiveness in Table 7-16. Because both MBRs and in-basin extended aeration systems meet current and anticipated standards without tertiary treatment, the additional cost to treat effluent from the extended aeration systems to meet municipal reuse standards have not been included in the estimate of project cost in Table 7-15. However a brief discussion of tertiary treatment including expected effluent quality and costs is included for completeness in the following section.

#### 7.6.1.1 Tertiary Treatment

Effluent complying with requirements for municipal reuse could be provided by installing a deep bed, granular media filter with continuous backwash following the in-basin extended aeration

system. This type of filter, with coagulation, flocculation, and separation within the sand bed, has been used for tertiary filtration for more than 10 years and several manufacturers have obtained validation for Title 22 Class A compliance.

Recently, advanced filtration system have been developed incorporating two stages of continuous up-flow sand filters. These systems remove contaminants to levels beyond the Class 2A requirements for municipal reuse to levels previously thought achievable only by membrane filtration. Advanced filtration systems are a low cost alternative to low pressure tertiary membrane systems or MBR treatment systems that incorporate enhanced biological and chemical treatment systems and can achieve the following effluent quality:

- Turbidity 0.05 -0.10 NTU (Nephelometric Turbidity Units)
- Phosphorous 0.01 - 0.05 mg/L
- Total Nitrogen < 1 mg/L
- Biological BOD < 3.0 mg/L

Estimated project costs for additional tertiary treatment:

- Sand filter module following in-basin extended aeration system to provide Class 2A Municipal Reuse: \$125,000 (each) providing 200 gpm (0.30 MGD)
- Two stage advanced filtration system following in-basin extended aeration system to provide the tertiary level of treatment cited above \$480,000 per MGD

## 7.6.2 Implementability

The constructability, adaptability, and future expandability of each of the alternatives were scored on a scale of 1 to 5, with 5 being the most favorable. Overall implementability was scored based on the weighted sum of each factor and is presented in Table 7-11.

<b>Table 7-11</b> <b>MCSD Secondary Treatment Alternatives Implementability</b> <b>MCSD Wastewater Management Facility</b>					
		Constructible	Adaptable	Expandable	Implementable
		Score	Score	Score	Weighted Score
1	HPAS <sup>1</sup> /Nitrifying Filter	3.0	2.0	1.0	2.0
2	In-Basin Extended Aeration	5.0	3.0	3.0	3.6
3	Oxidation Ditch	2.0	4.0	2.0	2.6
4	Activated Sludge with BNR <sup>2</sup> (MLE) <sup>3</sup>	2.0	4.0	4.0	3.3
5	MBR <sup>4</sup>	2.0	4.0	5.0	3.6
Weight		0.33	0.33	0.33	0.33
1. HPAS: High Performance Aeration System      3. MLE: Modified Ludzack-Ettinger (process) 2. BNR: Biological Nutrient Removal              4. MBR: Membrane Bioreactor					

All of the alternatives presented are constructed within the footprint of the existing basins, but Alternative 1, a high performance aeration system followed by a nitrifying filter and wetlands, has by far the largest construction footprint. Alternative 1 requires installation of baffles and new aerators, lining of the pond system and wetlands, construction of the nitrifying filter, and planting

of new wetlands. It is scored as less constructible than the in-basin extended aeration system (Alternative 2), but easier to construct than the oxidation ditch (Alternative 3), the MLE process, (Alternative 4), and the MBR (Alternative 5) because of the requirement for concrete construction of multiple components for these systems. Alternative 1 would be difficult to expand because incorporation of area outside the existing plant site would be required.

Scoring adaptability or operational flexibility is somewhat subjective, but included was the ability to take basins off line, add chemicals if required, bypass, and handle shock loading. All of the systems handle variable loadings and chemical addition would be possible for the extended aeration systems and membrane treatment. The oxidation ditch and MBR are more adaptable in terms of taking individual aerators, aeration basins, or clarifiers off line.

The in-basin extended aeration system and the MBR received identical scores for implementability. The in-basin system would be easier to construct and the membrane system (because of its modular construction) is more expandable. The membrane system was scored marginally higher on adaptability but this scoring did not take into account the increased complexity of MBR operation required to provide the operational flexibility.

### **7.6.3 Costs**

Construction costs for the five secondary treatment systems being evaluated were presented in the previous sections. To provide a basis for comparing total costs it was necessary to evaluate how annual operating cost would be affected by each alternative. The following factors will have an impact on operating costs:

- Aeration: annual power cost
- Operation and maintenance as reflected by increased staff
- Biosolids management

#### **7.6.3.1 Annual Power Cost**

Annual power requirements for the proposed alternatives are presented in Table 7-12.

<b>Table 7-12</b> <b>Annual Power Cost</b> <b>MCSD Wastewater Management Facility</b>					
			<b>Annual Usage</b>		
		<b>Horsepower</b>	<b>Operation</b>	<b>kWhr<sup>1</sup></b>	<b>Cost<sup>2</sup></b>
<b>Aeration</b>					
	Existing	60	60%	235,164	\$31,000
1	HPAS <sup>3</sup> /Nitrifying Filter	265	100%	1,731,068	\$226,000
2	In-basin Extended Aeration	150	70%	685,895	\$90,000
3	Oxidation Ditch	200	90%	1,175,820	\$153,000
4	Activated Sludge with BNR <sup>4</sup> (MLE) <sup>5</sup>	150	70%	685,895	\$90,000
5	Membrane Bioreactor	100	100%	653,233	\$85,000
<b>Pumping<sup>6</sup></b>					
	Existing			117,716	\$16,000
1	HPAS/Nitrifying Filter			183,039	\$25,000
2	In-basin Extended Aeration			248,363	\$34,000
3	Oxidation Ditch			215,701	\$29,000
4	Activated Sludge with BNR (MLE)			372,477	\$50,000
5	Membrane Bioreactor			529,253	\$71,000
<b>Aeration and Pumping</b>					
	Existing			418,203	\$47,000
1	HPAS/Nitrifying Filter			1,914,107	\$251,000
2	In-basin Extended Aeration			934,258	\$124,000
3	Oxidation Ditch			1,391,521	\$182,000
4	Activated Sludge with BNR (MLE)			1,058,372	\$140,000
5	Membrane Bioreactor			1,182,486	\$156,000
1. kWhr: Kilowatt hour 2. Power costs based on average of 0.13/kWh 3. HPAS: High Performance Aerated Pond System 4. BNR: Biological Nutrient Removal 5. MLE: Modified Ludzack-Ettinger (process) 6. Pumping costs for all alternatives include estimated \$16,000 per year for irrigation pumping					

The in-basin extended aeration system has the lowest annual power cost due to the following factors:

- Aeration system design is not mixing limited because of the configuration of the suspended aerators.
- Alternate aeration chains are operated on a timer to provide an anoxic zone for denitrification.
- Power requirements for auxiliary equipment are limited.

Power costs for the MBR system were based upon input from the manufacturer, and an assumption of complete mixing in the aeration basin preceding the membrane tank. Aeration costs for the MBR system are comparable to those of the in-basin extended aeration system, but overall, power costs are greater because of the cost of operating the permeate, or vacuum pumps, and other auxiliary equipment.



### 7.6.3.2 Biosolids Management Costs

Biosolids production for each of the secondary treatment alternatives is presented in Table 7-13. Yield estimates were based on the following values for volatile solids produced per pound of BOD:

- Existing = 0.20 lbs Volatile Suspended Solids (VSS)/lb BOD
- HPAS = 0.35 lbs VSS/lb BOD
- In-basin Extended Aeration = 0.45 lbs VSS/lb BOD- 4% further reduction in sludge storage
- Oxidation Ditch = 0.60 lbs VSS/lb BOD 4% further reduction in sludge storage
- Activated Sludge with BNR (MLE Process) = 0.70 lbs VSS/lb BOD
- Membrane Bioreactor = 0.70 lbs VSS/lb BOD

<b>Table 7-13</b> <b>Biosolids Management</b> <b>MCSD Wastewater Management Facility</b>											
Year	BOD <sup>1</sup> (ppd) <sup>2</sup>	Existing		HPAS/NF <sup>4</sup>		Extended Aeration		Ox Ditch		MBR <sup>5</sup> /MLE <sup>6</sup>	
		ton yr.	Annual Cost <sup>3</sup>	ton yr.	Annual Cost <sup>3</sup>	ton yr.	Annual Cost <sup>3</sup>	ton yr.	Annual Cost <sup>3</sup>	ppd	Annual Cost <sup>7</sup>
2010	2,234	82	\$81,541	103	\$102,742	178	\$177,938	252	\$251,612	1,944	\$439,832
2020	2,670	97	\$97,455	123	\$122,793	213	\$212,666	301	\$300,718	2,323	\$525,672
2030	3,191	116	\$116,472	147	\$146,754	254	\$254,163	359	\$359,398	2,776	\$628,247
1. BOD: Biochemical Oxygen Demand 2. ppd: pounds per day 3. Estimate based on \$1,000 per ton for private contractor 4. HPAS/NF: High Performance Aeration System/Nitrifying Filter 5. MBR: Membrane Bioreactors 6. MLE: Modified Ludzack-Ettinger 7. Estimate based on estimated cost of \$0.62/lb for hauling to Fortuna											

The aerated pond system is followed by partially mixed cells in which solids settle and undergo further digestion. For the purposes of this analysis, it has been assumed that the biosolids from the in-basin suspended aeration system and oxidation ditch system will be stored in a stabilization lagoon for approximately 10 years where they will undergo a similar process of digestion and a volatile solids reduction. Management costs for these three alternatives were based on a cost of \$1,000/dry ton to have biosolids removed from the storage lagoon dewatered and hauled to private disposal or land application sites.

The MLE process and MBR system produces a larger volume of less stabilized biosolids than the other secondary treatment alternatives, and long-term storage in a stabilization lagoon would not be feasible. It is assumed that the biosolids could be hauled to the wastewater treatment facility in Fortuna, which has a biosolids handling facility that will accept liquid biosolids. According to the superintendent of that facility, the charge per pound of biosolids would be approximately 60 cents and it is estimated that hauling would contribute an additional 2 cents.

### 7.6.3.3. Disinfection Cost

No significant deficiencies in the existing disinfection system were identified. The existing gas chlorination system is in good condition and in compliance with the Uniform Fire Code (fail safe valves are to be installed this year). The contact basin is of relatively new construction and has adequate volume to provide required contact times for projected peak day flows.

To determine whether an alternative disinfections system such as ultraviolet (UV) or on-site generation of hypochlorite could provide sufficient savings in annual operating costs to warrant consideration a present value analysis was performed. At the current average chlorine demand, annual costs for chlorine gas were greater than the estimated annual cost for on-site generation of hypochlorite but less than that for UV disinfection. The potential savings in annual cost did not provide enough of a return over the 20-year period calculated to pay for a new UV system at an estimated cost of approximately \$500,000.

The current average chlorine demand of greater than 10 mg/L is high due to interference from seasonally high levels of suspended solids and algae in the lagoon/wetlands effluent. Demand is expected to decrease with improved secondary treatment. At an average demand less than 7 mg/L annual operating costs are approximately equal to annual power and bulb replacement cost for UV and at the demands estimated for improved secondary treatment annual costs for chlorine are less than for either alternative evaluated.

Based on the lack of existing system deficiencies and the low cost of chlorine gas in comparison to other alternatives, annual costs for disinfection were estimated based on the existing system. Costs are presented in Table 7-14.

<b>Table 7-14</b> <b>Disinfection Annual Chemical Usage</b> <b>MCSD Wastewater Management Facility</b>											
	Chloride (Cl <sub>2</sub> ) Usage						Sulfide (SO <sub>2</sub> ) Usage				Chemical
	Demand	Residual	Dosage			Annual Cost	Dosage			Annual Cost	Annual Cost
	mg/L <sup>2</sup>	mg/L	mg/L	ppd <sup>3</sup>	t/yr. <sup>4</sup>		mg/L	ppd	t/yr		
Current	10.8	2.6	13.4	117	21	\$12,615	4	40	7	\$4,551	\$17,167
Alt 1	7	2	9	90	16	\$9,699	3	35	6	\$3,952	\$13,650
Alt 2	6	1	7	70	13	\$7,543	2	17	3	\$1,976	\$9,519
Alt 3	5	1	6	60	11	\$6,466	2	17	3	\$1,976	\$8,441
Alt 4	4	1	5	50	9	\$5,388	2	17	3	\$1,976	\$7,364
Alt 5	3	1	4	40	7	\$4,310	2	17	3	\$1,976	\$6,286
1. mg/L: milligrams per Liter 2. ppd: pounds per day 3. t/yr.: tons per year											

### 7.6.3.4 Project Cost/Present Value

Annual operating costs are summarized in Table 7-15. The 20-year present value of these costs was then added to the Engineer's estimate of probable project cost to obtain a present value estimate for comparison of secondary treatment alternatives.

In addition to the annual operating costs shown in Table 7-15, there may be costs associated with the occasional addition of caustic soda or lime to increase alkalinity in the system. The potential cost for this application would be minimal and would apply to all alternatives; therefore this cost has not been included for comparison purposes.

<b>Table 7-15</b> <b>Comparison of Annual Costs by Alternative and Project Present Value</b> <b>MCSD Wastewater Management Facility</b>						
Alternative	Current	HPAS/NF <sup>1</sup>	Extended Aeration	Oxidation Ditch	MLE <sup>2</sup>	MBR <sup>3</sup>
Power Costs <sup>4</sup>	\$47,000	\$251,000	\$124,000	\$182,000	\$140,000	\$156,000
Biosolids Management	\$97,455	\$122,793	\$212,666	\$300,718	\$525,672	\$525,672
Chlorine/Sulfide	\$17,167	\$13,650	\$9,519	\$8,441	\$7,364	\$6,286
Personnel	\$50,000	\$50,000	\$100,000	\$100,000	\$100,000	\$100,000
Annual Training	-	-	\$3,000	\$3,000	\$3,000	\$6,000
Annual Ops Costs	\$211,622	\$437,443	\$449,185	\$594,160	\$776,036	\$793,958
Ops Present Va. <sup>5</sup>	\$2,877,000	\$5,945,000	\$6,105,000	\$8,075,000	\$10,547,000	\$10,791,000
Project Cost	---	\$7,270,000	\$7,427,000	\$8,686,000	\$10,975,000	\$14,156,000
Project Present Value	---	\$13,215,000	\$13,532,000	\$16,761,000	\$21,522,000	\$24,947,000
1. HPAS/NF: High Performance Aerated Pond System/Nitrifying Filter 2. MLE: Modified Ludzack-Ettinger 3. MBR: Membrane Bioreactors 4. Power cost based on an average of \$0.13/kWh 5. 20-Year present value, discount rate of 4%						

## 7.6.4 Preferred Project

Secondary treatment alternatives with the capacity to treat projected loadings and produce a high quality effluent complying with requirement for discharge to the Mad River in wet weather and land reclamation or disposal during the summer have been evaluated with regard to treatment, cost, and implementability. As shown in Table 7-16, the in-basin extended aeration system can meet anticipated permit requirements at the lowest cost.

<b>Table 7-16</b> <b>Summary Evaluation Matrix</b> <b>MCSD Wastewater Management Facility</b>							
Alternative		Total <sup>1</sup>	Reliability/ Effect.	Implement -ability	Cost	Public Acc.	Reg. Issues
1	HPAS/NF <sup>2</sup>	4.0	1	2	5	3	4
2	Extended Aeration	4.9	3	3.6	5	3	5
3	Oxidation Ditch	3.7	3	2.6	3	2	5
4	Activated Sludge (MLE Process) <sup>3</sup>	3.7	4	3.3	2	2	5
5	MBR <sup>4</sup>	3.7	5	3.6	1	3	5
Weight <sup>5</sup>			0.2	0.2	0.4	0.2	0.2
1. High score considered to be most favorable. 2. HPAS/NF: High Performance Aerated Pond System/Nitrifying Filter 3. MLE: Modified Ludzack-Ettinger 4. MBR: Membrane Bioreactors 5. Higher cost weight includes project cost and operations & maintenance as a measure of operational complexity							

There are no regulatory or public acceptance issues anticipated regarding construction because the facility upgrades will be constructed within the footprint of the existing plant.

## 7.7 Biosolids Management

The extended aeration system recommended as the preferred alternative for improvements to secondary treatment at the McKinleyville facility produces a much stabilized sludge. The volume of sludge is also lower than other activated sludge processes making it feasible to store the biosolids for periodic removal.

Biosolids management alternatives appropriate for the WWMF include the following:

- Long-term storage in sludge stabilization pond (assumes contracting of solids removal on 10-year basis)
- Land application of liquid biosolids on MCSD land
- Contract with City of Fortuna for disposal

### 7.7.1 Long-Term Storage

Average annual biosolids production was calculated based upon projected BOD loadings. Sludge yield for the extended aeration process without primary clarification was estimated to be approximately 0.45 lbs total solids/lb BOD removed (Metcalf and Eddy, 1991 Table 12-7). If the sludge is stored for longer than several months, the volatile fraction is reduced by an estimated 40%. The results of these calculations are presented in Table 7-17.

<b>Table 7-17</b> <b>Biosolids Storage</b> <b>MCSD Wastewater Management Facility</b>									
Year	BOD <sup>1</sup>	Sludge Yield <sup>3</sup>		Stored <sup>5</sup>	Annual Production		Sludge Storage <sup>7</sup>		
	ppd <sup>2</sup>	VSS <sup>3</sup>	TSS <sup>4</sup>	TSS	TSS	TSS	Volume		Area
		ppd	ppd	ppd	ton	MG <sup>6</sup>	MG	ac.ft. <sup>8</sup>	Acres <sup>9</sup>
2010	2,234	1,005	1,377	975	178	0.70	7	22	2.0
2020	2,670	1,202	1,646	1,165	213	0.86	9	27	2.4
2030	3,191	1,436	1,967	1,393	254	1.03	10	32	2.9
1. BOD: Biochemical Oxygen Demand 2. ppd: pounds per day 3. Sludge yield 0.45 lb, Volatile Suspended Solids (VSS)/lb BOD, Volatility 70% 4. TSS: Total Suspended Solids 5. Volatile fraction of sludge reduced by 40% 6. MG Million gallons at 6% 7. Required storage 10-years 8. ac.ft.: acre feet 9. Assumes 11 feet of depth									

The sludge storage area shown in Figure 7-2 is approximately 2 acres and can hold approximately 6 MG of sludge assuming an 11-foot depth and 2-foot water cap. This volume will provide approximately 9 years of storage at current loading rates and 8 years at projected 2030 loading rates. Estimated costs to employ an independent contractor for biosolids removal and disposal are approximately \$1,000 per dry ton. The estimate is based upon recent biosolids removal cost for similarly sized municipalities in the area.

## 7.7.2 Land Application

Biosolids management cost could be significantly reduced if liquid biosolids could be land applied, especially if land used by the MCSD for disposal of treated effluent could also be employed for biosolids disposal. To determine the feasibility of this approach, the nitrogen contributed by the biosolids was evaluated and the area required for land application based on an assumed loading rate for nitrogen of 120 lbs/acre/year. The results are summarized in Table 7-18.

<b>Table 7-18</b> <b>Land Application of Biosolids</b> <b>MCSD Wastewater Management Facility</b>						
Year	TSS <sup>1</sup>	NH <sub>4</sub> -N <sup>2</sup>	NO <sub>3</sub> -N <sup>4</sup>	Org-N <sup>5</sup>	PAN <sup>6</sup>	Acres
	ton/year	lbs/year <sup>3</sup>	lbs/year	lbs/year	lbs/year	
2010	178	1,763	35	2,115	3,913	33
2020	213	2,163	43	2,596	4,803	40
2030	254	2,586	52	3,103	5,741	48
1. TSS: Total Suspended Solids 2. NH <sub>4</sub> -N: Ammonium-Nitrogen (assumed to be 1% of total solids availability 50%) 3. lbs/year: pounds per year 4. NO <sub>3</sub> -N: Nitrate-Nitrogen (assumed to be 0.1% of total solids availability 100%) 5. Org-N: Organic Nitrogen (assumed to be 2% of total solids availability 30%) 6. PAN: Plant-Available Nitrogen (based on agronomic loading rate for nitrogen of 120 lbs/acre/year)						

## 7.7.3 Hauling

The City of Fortuna wastewater treatment plant is accepting liquid biosolids for treatment and composting. The biosolids handling facility is currently making a Class A composted biosolid used for landscaping by local residences, businesses, and nurseries. Currently, charges range from \$0.60/dry lb for highly volatile biosolids to \$0.80/lb for more stabilized biosolids.

The liquid biosolids could be hauled by truck and discharged into the digester at Fortuna on a biweekly basis, eliminating the need for the large sludge storage lagoon. There would be an increase in the volume of biosolids handled because digestion in the sludge lagoon and the resulting reduction in volume would be eliminated. However, the increased volume at current rates charged by Fortuna for handling and composting would not be cost effective for MCSD.

Because of a lack of biosolids disposal options within Humboldt County, Fortuna is being encouraged by the RWQCB to handle biosolids on a regional basis. If other communities take advantage of this service, the rates may be reduced, making this a more cost-effective solution for MCSD.

## 8.0 Disposal and Reclamation Alternatives

MCSD is currently permitted to discharge treated wastewater effluent to the Mad River (Discharge Point 001) from October 1 through May 14 (the discharge period), if river flows are greater than 100 times the wastewater flow and the flow in the river is greater than 200 cubic feet per second. If the flow conditions are not met, effluent is discharged to the percolation ponds adjacent to the river (Discharge Point 002) and/or to land for reclamation (use as irrigation water). From May 15 through September 30 (the discharge prohibition period), effluent is discharged to the percolation ponds (Discharge Point 002) and/or to land for reclamation. Discharge to land occurs at the Lower Fisher Ranch (Discharge Point 003), Upper Fisher Ranch (Discharge Point 004), the Hiller Parcel (Discharge Point 005), and the Pialorsi Ranch (Discharge Point 006). The existing discharge points are shown on Figure 5-1.

A series of disposal and reclamation alternatives have been evaluated that will allow MCSD to comply with discharge regulations under existing and projected flow conditions, including:

- New reclamation practices
- Continued use of the existing outfall to the Mad River
- Municipal Reuse
- Use of an Ocean Outfall

### 8.1 New Reclamation Practices

Under current conditions wastewater reuse on the existing wastewater reclamation areas does not conform to the current waste discharge requirements for reclamation activities. The Upper Fisher Ranch is not currently operated for reclamation; wastewater effluent is applied by overland flow irrigation methods in quantities that exceed agronomic rates for hay grass. Opportunities to increase irrigation on the lower pastures may balance these effects; however, based on current nitrogen loading rates, the existing available reclamation area is not sufficient to reclaim wastewater. The existing percolation ponds are also proposed to be removed from service. In order to accommodate the land application of effluent, modifications to the existing disposal management practices will need to include a reduction in total nitrogen in the plant effluent and an increase of the crop cover's ability to use the available nitrogen being applied through land application.

To increase reclamation capabilities at the land reclamation sites, installation of a poplar forest is proposed. The agronomic rate for the existing crop cover allows for the application of a total nitrogen loading of 120 to 170 pounds of nitrogen per acre per year. Poplars (*Populus* spp.) have proven to be effective biofilters and their forests provide a cost-effective method to recycle nutrients from wastewater discharge (EPA 2006). Literature suggests that poplars have a high transpiration rate and can have an average nitrogen uptake of 270 pounds of per acre per year, for a whole tree harvesting cycle of 6 to 15 years. By replacing the existing crop with a poplar tree forest, the total nitrogen loading can be increased from the current 170 pounds per acre per year to 220 -250 pounds per acre per year, reducing the total acreage required for land application.

Poplars are deciduous hardwood trees that grow in a wide variety of climates and soil conditions. There are four primary species of poplars that are used to create fast growing hybrids. Those include *Populus deltoids* (Eastern Cottonwood), *Populus nigra* (European Black Poplar), *Populus trichocarpa* (Western Black Cottonwood) and *Populus maximowiczii* (Asian Poplar).

Poplar trees are becoming a preferred treatment for the reuse of municipal and industrial wastewater. The trees have been proven to clean the water effectively and often provide a cheaper alternative than building additional treatment facilities. Poplar trees used for wastewater reuse in many instances will pay for themselves at harvest when the wood fiber is used for lumber, paper, or fuel for bio-energy.

The increased interest in using poplar trees in the treatment of wastewater is in part due because they are easy to manage and can provide a variety of secondary beneficial uses and products. A few of these products include fiber for pulp and paper, high quality lumber, poplar wood chips, biomass for renewable energy, shade, and windbreaks. In addition, poplar trees are one of the fastest growing trees. Poplars take less than 15 years to mature, whereas most other trees take 15 to 50 years to reach full maturity. This is an important factor for the following reasons: first, mature trees require more water for survival and growth; therefore, more wastewater can be applied as the trees mature. Secondly, older trees generally provide better quality lumber than do younger trees. Poplar trees also have a relatively high water uptake rate. Average poplar trees, aged at 3 years can absorb up to 10,000 gallons of water per acre per day in summer months.

By using plant systems for reclamation of treated effluent, additional benefits of improved air quality are provided. Trees and shrubs have the potential to be an effective and inexpensive odor control mechanism. Trees can induce deposition of particulate matter by reducing wind speeds, and tree leaves can remove gaseous pollutants from the atmosphere. Trees also remove carbon dioxide (CO<sub>2</sub>), one of the primary greenhouse gases that cause global warming.

### 8.1.1 Description of Disposal Strategy

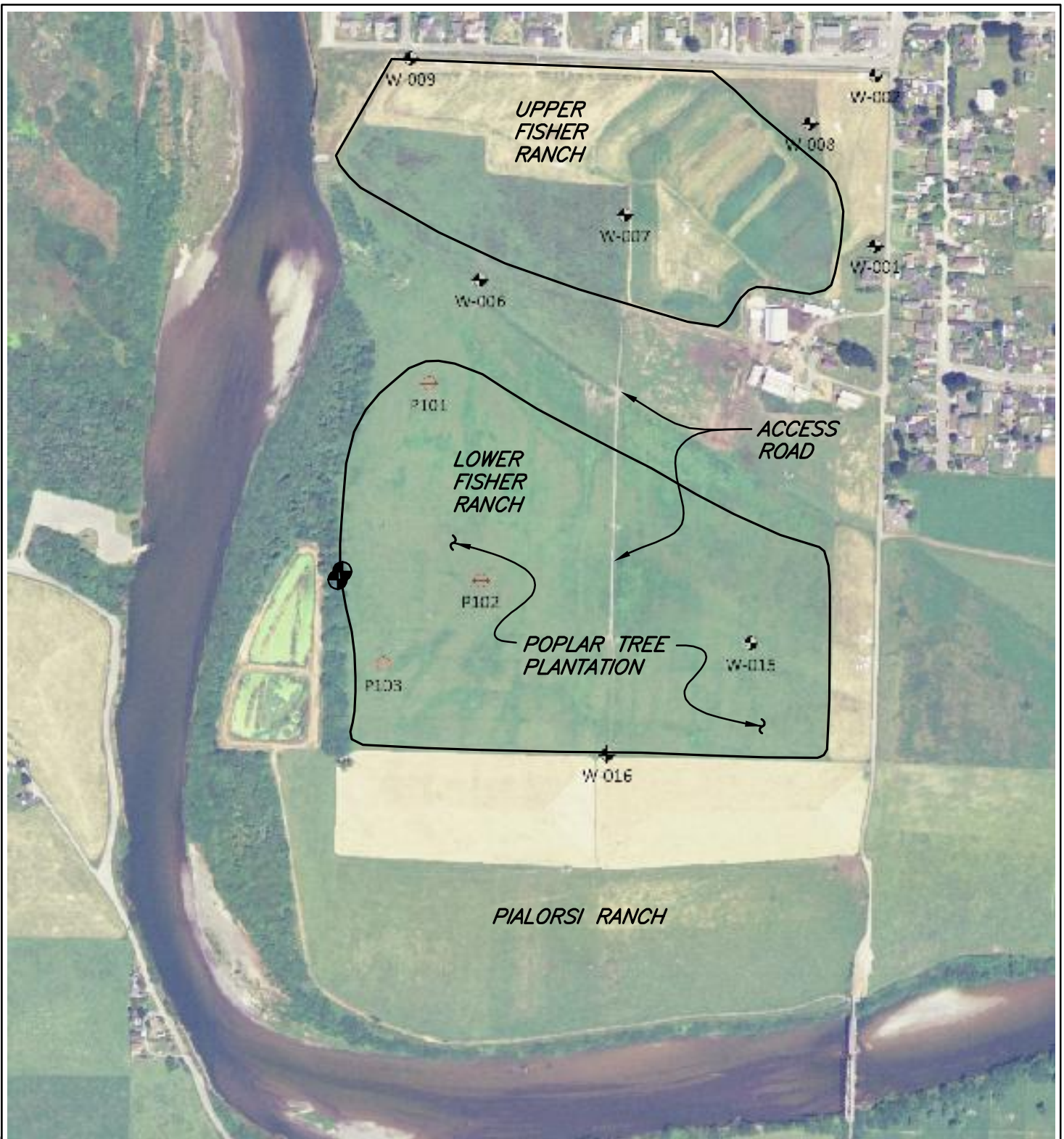
#### 8.1.1.1 Poplar Forest

The proposed poplar forest disposal plan includes the planting of a minimum of 45 acres of the lower Fisher Ranch property with poplars in 4 to 5 acre plots (Figure 8-1). The trees will receive wastewater through the irrigation system at a flow rate based on the age of the trees in the individual plot. Each plot of trees will be allowed to mature to the age of 10 to 15 years. The total acreage of the poplar forest is to be determined based on the results of the on-going pilot study. Additional acreage of trees will provide for the potential application of biosolids as part of a diverse biosolids disposal plan.

As the trees reach maturity, individual plots will be harvested in a crop rotation, harvesting no more than 10% of the forest in any single year. The rotation of the plots is designed to provide the maximum amount of mature trees while the younger trees have the opportunity to develop. The age of the trees at harvest can be varied depending on the end use of the harvested trees—the use of the trees for biomass generation requires a much shorter life cycle than trees being harvested for commercial milling.

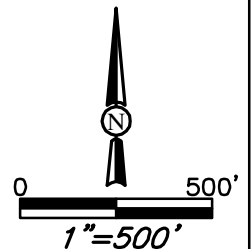


\\Zing\projects\2008\008189-MCSD\300-Facilities\Plan\Drawings , SAVED: 8/9/2011 5:14 PM NDOWNNEY, PLOTTED: 1/11/2012 3:08 PM , NATHAN DOWNEY



## EXPLANATION

● MONITORING WELL



Each plot of trees will be irrigated with wastewater at agronomic rates adjusted for the age of the trees in each plot. A variety of irrigation systems will be evaluated with the evaluation criteria to include cost of installation, reliability, flow characteristics, and operation and maintenance considerations. The implementation of the preferred treatment plant modifications will reduce the total nitrogen from the current levels of 18 to 28 mg/L to total nitrogen of less than 8 mg/L. This reduction in total nitrogen will produce an effluent with an available nitrogen level of less than 6 mg/L.

Maintaining the existing crops and the current average annual flow of 1.12 MGD (2010) a total of 217 acres are required for land reclamation, assuming available nitrogen is equal to the current average annual TKN loading rate of 35 mg/L (2010). If poplars replaced the current grass crop mixture on the lower Fisher Ranch property, total acreage efficiency could be increased by 130%. Correspondingly, after future upgrades to the facility reduce available nitrogen concentrations to 6 mg/L and assuming an annual ADWF of 1.4 MGD, the total acreage required will be reduced from 90 acres (based on the current crop), to 75 acres, using poplars on approximately 45 acres of the available land reclamation sites.

Advantages:

- Ability to land apply entire dry weather flows without the use of the percolation ponds additional storage.
- Incorporation of biosolids disposal in the early years and potential for future use
- Low cost for development of alternative crops
- Biomass or merchantable timber from the harvesting of mature trees
- Flexibility to manage disposal based on variable weather conditions

Disadvantages:

- Additional operations and maintenance requirements over existing practices
- Increased nutrient management and recording required
- Groundwater monitoring requirements

### 8.1.1.2 Cost

The implementation of a poplar forest disposal system will be spread over a 10-year period in order to build in the required rotation of the plants for harvesting. The initial planting would consist of 20 to 25 acres with the remaining acres being installed 5 acres per year for the following 4 to 5 years. The budgetary cost estimates presented in Table 8-1 are based on the construction of 45 acres of poplar forest, which is the minimum required acreage. The cost for the development of the poplar forest would be spread over a five to six year period.

<b>Table 8-1</b> <b>Engineer's Opinion of Probable Cost for Poplar Forest</b> <b>MCSO Wastewater Management Facility</b>				
Description	Unit	Unit Cost	Quantity	Total Cost
Mobilization 12%				\$121,500
Site Preparation	ac <sup>1</sup>	\$500	45	\$22,500
Planting of Trees	ac	\$2,000	45	\$90,000
Irrigation System	ac	\$20,000	45	\$900,000
Construction Subtotal				\$1,134,000
Contingency 20%				\$226,800
Engineering 25%				\$283,500
Admin 4%				\$45,360
<b>Project Subtotal</b>				<b>\$1,689,660</b>
1. ac: acre				

Harvesting activities will begin around year 12 after the initial planting. Costs associated with harvesting are not available at this time, but it is generally acknowledged that the cost of harvesting and replanting can be offset with the sale of the harvested trees as a bio-fuel or potentially for commercial milling. Operations and maintenance costs are projected to be approximately \$25,000 per year to maintain the irrigation system.

### 8.1.2 Poplar Tree Pilot Study

A pilot study area has been established to evaluate the efficiency of poplar tree nutrient uptake in the southwest corner of the Lower Fisher Ranch reclamation area. The 1-acre site uses the existing irrigation main line that distributes wastewater effluent to the percolation ponds. Access to the pilot forest is from Fisher Avenue and the ranch roads that traverse the pastures. The pilot forest site is situated on the landscape to allow for expansion to the north and east.

A variety of trees, including hybrid poplars (black cottonwood and eastern cottonwood cross-fertilized), as well as trees native to the north coast of California were evaluated for consideration for the pilot study. The Black cottonwood (*Populus trichocarpa*) is a native cottonwood that was locally available and was selected for use based on its ability to uptake large quantities of water and its high nutrient assimilation capacity. Additional selection criteria included tree growth characteristics, viability, potential for beneficial use of harvested material, and local availability.

## 8.2 Existing Outfall to Mad River

During the discharge season, which extends from October 1 through May 14, wastewater is discharged from Discharge Point 001 to the Mad River. Discharge to the river is contingent on the flow in the river being above 200 cfs. During dry years, land application continues into the late summer and early fall months due to low river levels.

The existing discharge consists of a 16-inch pipe that extends from the treatment plant to the outfall located at the Hammond Trail Bridge crossing on the Mad River. The 16-inch plant effluent piping is reduced to an 8-inch pipe with a 16-foot length of flexible rubber pipe at the outfall. As the plant flows increase, the 8-inch piping and flexible hose may require upsizing; the capacity will need to be verified during additional pre-design efforts.

## 8.3 Municipal Reuse

It is recommended that the MCSD solicit public input regarding implementation of a municipal reuse program. Disinfected tertiary recycled water can be used for local schools, parks, golf courses, and so on. The main advantage of using recycled water is that it reduces peak demands on the municipal water system and storage tanks maintained by MCSD.

### 8.3.1 Requirements

Disinfected tertiary recycled water requires that secondarily treated wastewater is filtered prior to disinfection and that disinfection meets either of the following criteria:

- A CT (the product of chlorine residual and contact time ) value greater than 90 minutes
- A median concentration of total coliform less than 2.2 MPN/100 ml and a maximum concentration of 23 MPN/100 ml

### 8.3.2 Implementation

The in-basin extended aeration system recommended in Section 7 as an upgrade to secondary treatment at the WWMF will provide disinfected secondary, 23 MPN recycled water suitable for irrigation and reclamation. A portion of the secondary effluent would be filtered using a granular media filters and the filtered water would be disinfected as separate side stream to provide disinfected tertiary recycled water.

The least expensive granular media filters are continuous backwash filters with an upflow configuration. Budgetary costs are estimated to be \$480,000 per MGD. In addition to chemical addition and filtration systems, total project cost would include a separate purple pipe pumping and distribution system and would depend upon the demand for recycled water.

## 8.4 Ocean Outfall

The feasibility of year-round disposal of McKinleyville wastewater effluent using an ocean outfall was investigated because if it could be permitted, an ocean outfall could have significant advantages:

- It would eliminate the need for land reclamation, resulting in significant operations and maintenance savings
- If the mixing zone is approved, permit requirements for ammonia would be less stringent than they are for Mad River discharge, resulting in reduction in treatment costs.

- Concerns regarding the variable location of the river mouth and the associated extent of the adjacent estuary in the vicinity of the existing discharge location would be eliminated.

### 8.4.1 Regulatory Issues

A summary of issues regarding the proposed ocean outfall investigation was prepared by SHN in September 2010 (SHN, 2010). A copy of the summary of issues is included in Appendix H.

Two pre-application meetings were conducted in October and November 2010 to initiate the permitting process for construction of an ocean outfall for the WWMF. Representatives from the RWQCB, California Coastal Commission (CCC), California Department of Fish and Game (CDFG), National Marine Fisheries Service, US Army Corps of Engineers, and California State Lands Commission (CSLC) attended the meetings. A copy of the meeting summary from each workshop is also included in Appendix H.

Follow up discussion with CCC representatives was conducted in March 2011 to determine the level of effort needed for further permitting review of the ocean outfall alternative. The CCC indicated that the permitting of a new ocean outfall would require completion of an alternatives analysis that looked at other feasible alternatives for disposal that are not coastal dependant. If the analysis determines that other feasible options do not exist for disposal, then the CCC would consider pursuing permitting of the project. However if other onshore alternatives are available for disposal, the District would be directed to pursue the onshore alternatives over construction of a new ocean outfall.

Ocean outfalls in California are only permitted on a case-by-case basis. Regulatory agencies have indicated that a new ocean outfall would most likely not be permittable if other disposal alternatives exist. When coupled with improved secondary treatment and significant nitrogen reduction, continued surface water discharge to the Mad River and land reclamation at agronomic rates will comply with current and anticipated regulatory constraints.

The following information is presented herein to give the MCSD a basis for comparison to the current disposal strategy and to provide an alternative for disposal if regulations regarding discharge to the Mad River outfall change.

#### 8.4.1.1 Mixing Zones

The EPA defines a mixing zone as an allocated impact zone where water quality standards may be exceeded as long as acutely toxic conditions are prevented and the State's beneficial uses are protected. Use of regulatory mixing zones as defined by the EPA is allowed at the discretion of the State. Historically, the RWQCB has decided whether to allow mixing zones in ocean on a case-by-case basis.

If a Regulatory Mixing Zone (RMZ) is permitted, water quality criteria must be met at the edge of the mixing zone. Water quality objectives for protection of marine life are defined in Table B of the California Ocean Plan. Constituents of particular concern in the McKinleyville effluent would include copper, lead, ammonia, diethyl phthalate, 4,4'-DDT, and dioxin equivalents.

Within RMZ A, a Toxic Dilution Zone (TDZ) may be provided to allow for dilution of toxic constituents below acute criteria or Criteria Maximum Concentration (CMC). The EPA provides guidance for setting stringent criteria that can be used to limit the TDZ based on probable exposure with no impact from short-term contact with toxic constituents. Three criteria are provided below, the more stringent of which should govern the limit of the TDZ.

1. The CMC should be met at a location that is 10% of the distance from the edge of the outfall structure to the edge of the RMZ.
2. The CMC should be met within a distance of 50 times the discharge length scale in any special direction. (The discharge length scale is calculated as the square root of the discharge port area.)
3. The CMC should be met within a distance of five times the local water depth in any horizontal direction from any discharge outlet.

#### **8.4.1.2 Secondary Treatment**

A new NPDES permit will need to be issued for an ocean outfall. Under the Clean Water Act (CWA), wastewater discharges from Publicly Owned Treatment Works (POTWs) are required to receive at least secondary treatment, unless an "ocean waiver" is obtained. Given anti-degradation requirements, treatment equivalent to current waste discharge requirements would be the minimum granted even under such a waiver.

It is assumed that secondary treatment limits for BOD and TSS of 30 mg/L will apply. Improvements to secondary treatment will be required because the existing facultative treatment system is at capacity for BOD removal. Secondary treatment could be provided by any of the secondary treatment alternatives discussed in Section 7.

#### **8.4.1.3 Ammonia**

The need to implement advanced treatment for nitrogen removal to provide effluent that complies with water quality limits for  $\text{NH}_4\text{-N}$  protective of aquatic life in the marine environment, and will depend upon whether the RWQCB permits a TDZ. Acute toxicity limits or CMC must be met within the TDZ, and Chronic or Continuous Concentration (CoCC) within the RMZ.

The CoCC and CMC vary with pH, temperature, and salinity, and can be calculated based on some assumptions regarding those parameters (Marshack, 2011). Estimates of these criteria and the required dilutions assuming secondary treatment without advanced levels of nitrogen removal are presented in Table 8-2.

<b>Table 8-2</b> <b>Ocean Outfall Ammonia Limits</b> <b>MCSO Wastewater Management Facility</b>						
	Criteria	Marine Environment <sup>3</sup>			Effluent	Dilution Required
	NH <sub>4</sub> -N <sup>1</sup>	pH	Temp	Salinity	NH <sub>4</sub> -N	
	mg/L <sup>2</sup>	(pH units)	°C	g/kg <sup>4</sup>	mg/L	
CMC <sup>5</sup>	6.0	8.1	15	20	20	3.3
CCC <sup>6</sup>	1.0	8.1	15	20	20	20
1. NH <sub>4</sub> -N: Ammonium-Nitrogen 2. mg/L: milligrams per Liter 3. Limiting values for ambient parameters 4. g/kg: grams per kilogram 5. CMC: Criteria Maximum Concentration (1-hr. average) 6. CoCC; Criteria Continuous Concentration (4 day average)						

#### 8.4.1.4 Required Treatment

If a toxic mixing zone is allowed for ammonia, enhanced treatment for nitrogen removal may not be required. Secondary treatment could be provided by an HPAS without a nitrifying filter at an estimated project cost of \$4,700,000, or by treatment wetlands at an estimated project cost of \$6,000,000.

### 8.4.2 New Outfall

The feasibility of constructing a new ocean outfall was discussed with the District and pertinent regulatory agency representatives in October and November 2010, with subsequent discussions with CCC staff in March 2011. A summary of the information reviewed is included in Appendix H.

#### 8.4.2.1 Alignment

Locating a new outfall would depend upon many parameters, including:

- suitability for Horizontal Directional Drilling (HDD),
- offshore bathymetry, and
- subsurface conditions near the outfall diffusers.

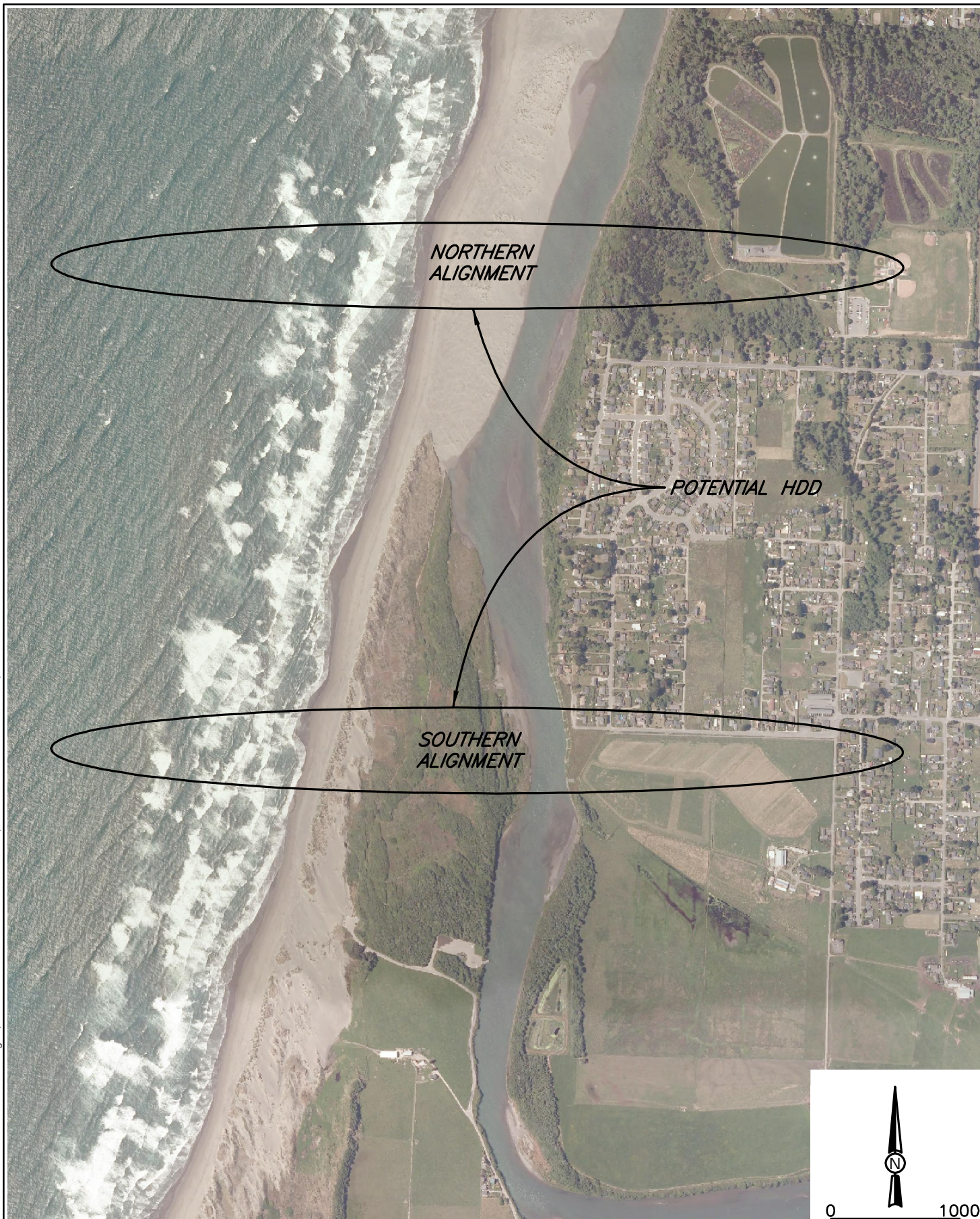
It is assumed that the outfall would be constructed 2,000 to 4,000 feet offshore. This would put it outside of the zone of “immediate contact,” which is defined in the ocean plan as the zone bound by the shoreline and a distance of 1,000 feet, or the 30-foot depth contour, whichever is less. In preliminary discussions two alignments were considered and are presented in Figure 8-2.

#### 8.4.2.2 Budgetary Cost

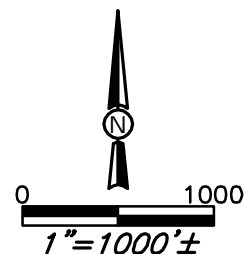
Estimates of probable cost for the two alignments are presented in Table 8-3. This is a budgetary estimate and because of the uncertainty involved, a contingency of 30% is included in the estimate of project cost.



\\Eureka\Projects\2008\008189-MCSD\300-FacilitiesPlan\Drawings\SAVED: 8/5/2011 2:23 PM CNEWELL, PLOTTED: 8/8/2011 8:47 AM, CHRIS D. NEWELL



SOURCE: 2010 NAIP IMAGERY





<b>Table 8-3</b> <b>Ocean Outfall Estimate of Probable Cost</b> <b>MCSD Wastewater Management Facility</b>				
<b>Description</b>	<b>Unit</b>	<b>Unit Cost</b>	<b>Quantity</b>	<b>Total Cost</b>
<b>Northern Alignment</b>				
Mobilization 12%				\$541,800
HDD <sup>1</sup>	LF <sup>2</sup>	\$1,000	4,000	\$4,000,000
Diffusers	LS <sup>3</sup>	\$75,000	ALL	\$75,000
18-inch gravity discharge	LF	\$220	2,000	\$440,000
Construction Subtotal				\$5,056,800
Contingency 30%				\$1,517,040
Engineering 25%				\$1,264,200
Admin 4%				\$202,272
<b>Project Subtotal</b>				<b>\$8,040,312</b>
<b>Southern Alignment</b>				
Mobilization 12%				\$580,200
HDD	LF	\$1,000	3,000	\$3,000,000
Diffusers	LS	\$75,000	ALL	\$75,000
18-inch gravity discharge	LF	\$220	8,000	\$1,760,000
Construction Subtotal				\$5,415,200
Contingency 30%				\$1,624,560
Engineering 25%				\$1,353,800
Admin 4%				\$216,608
<b>Project Subtotal</b>				<b>\$8,610,168</b>
1. HDD: Horizontal Directional Drilling 2. LF: Linear Foot 3. LS: Lump Sum				