# CITY OF MANTECA

# WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN UPDATE

REGISTERED PROFESSIONAL ENGINEER
No. 33479
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CIVIL
STATE OF CALIFORNIA

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Submitted to:

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The following abbreviations are used in this report:

| | |
|---|---|
| ADWF | average dry weather flow |
| BDCM | Bromodichloromethane |
| BOD | Biochemical Oxygen Demand |
| BP | Basin Plan |
| CIP | capital improvement plan |
| CS | combined sludge (primary sludge and thickened waste activated sludge) |
| CTR | California Toxic Rule |
| DAF | Dissolved air flotation |
| DHS | California Department of Health Services |
| DO | dissolved oxygen |
| DWR | Department of Water Resources |
| EC | electrical conductivity |
| HRT | hydraulic retention time |
| I-5 | Interstate 5 |
| IPP | Industrial Pipeline Project |
| IPS | influent pump station |
| LAA | Land Application Area |
| MBAS | Methylene Blue Active Substances |
| MCL | maximum contaminant level |
| MCRT | mean cell residence time |
| MDF | maximum daily flow |
| mgd | million gallons per day |
| MHF | maximum hourly flow |
| MLE | modified Ludzack-Ettinger |
| MLSS | mixed liquor suspended solids concentration |
| MUN | Municipal |
| NSF | Northside facilities |
| NTU | nephelometric turbidity unit |
| NPDES | National Pollutant Discharge Elimination System |
| NWRI | National Water Research Institute |
| OEHHA | California Office of Environmental Health Hazard Assessment |
| PFIP | Public Facilities Implementation Plan |
| PPP | pollution prevention programs |
| PS | Primary sludge |
| PSB | primary sedimentation basin |

| | |
|---|---|
| RAS | recycled activated sludge |
| RWQCB | Regional Water Quality Control Board |
| SIP | State Implementation Policy |
| SR-120 | State Route 120 |
| SSF | Southside facilities |
| SSJID | South San Joaquin Irrigation District |
| TMDL | total maximum daily load |
| TN | total nitrogen |
| TSS | Total Suspended Solids |
| TTHM | Total trihalomethanes |
| TWAS | thickened waste activated sludge |
| UMP | updated master plan |
| VSS | volatile suspended solids |
| WAS | waste activated sludge |
| WGF | Wastewater generation factors |
| WQCF | Manteca Wastewater Quality Control Facility |

# 1 Introduction

In support of a new General Plan and updated Public Facilities Implementation Plan (PFIP), an updated wastewater treatment and disposal master plan is required. Background information, the scope of the master plan update, facility operational history, a description of the treatment facility, and a discussion of the Phase III Expansion Project are presented in this chapter.

## 1.1 Background and Purpose

The Manteca Wastewater Quality Control Facility (WQCF) is a 6.95 million gallons per day (mgd) rated combined biofilter-activated sludge plant. Secondary effluent is land applied during the spring and summer (flood irrigation for agricultural production) and discharged to the San Joaquin River during the winter (October-March). In the future, year-round discharge to the river is anticipated because of the limited capacity of City-owned land for wastewater applications. In conjunction with the development of the City of Manteca Public Facilities Implementation Plan (PFIP), a master plan for the Manteca WQCF was prepared in 1993 [1]. The master plan included an analysis of existing unit processes, an evaluation of expansion options for the facility, a discussion of disposal alternatives, and a description of phased improvements. The Phase III Expansion Project incorporates the recommendations of the 1993 Master Plan and reflects the current needs and operating requirements for the plant. The capacity of the Manteca WQCF will increase from 6.95 mgd to 9.87 mgd (average dry weather flow) following completion of the Phase III expansion project in 2007. This capacity is anticipated to support City-wastewater requirements for approximately 5-10 years.

Beyond the Phase III expansion, the City has identified the need to program future facilities to accommodate average dry weather flows (ADWF) up to 27 mgd (buildout condition). In addition, the existing land outfall to the San Joaquin River will reach capacity following completion of the Phase III expansion project. Any expansion beyond a capacity of 9.87 mgd will require the construction of a second outfall. In addition, future river discharge requirements are expected to become stricter and may entail additional treatment beyond effluent filtration. In view of these developments, an updated master plan (UMP) that considers stricter water quality requirements, alternative disposal options, and increased flows is warranted.

## 1.2 Scope of Master Plan Update

The objective of the UMP is to determine what steps will be required to increase the treatment capacity from approximately 10 mgd to 27 mgd while complying with stricter effluent discharge requirements for river discharge. More specifically, the UMP includes the following:

1. A summary of probable future water quality requirements for continued discharge to the San Joaquin River;

2. An assessment of alternative wastewater disposal options with an emphasis on either urban irrigation or expanded agricultural irrigation program;

3. A description of proposed treatment plant improvements for both river and land-based wastewater disposal strategies; and,

4. A detailed capital improvement plan (CIP) that provides appropriate infrastructure to support growth while ensuring compliance with stricter effluent discharge requirements.

## 1.3 Facility Operational History

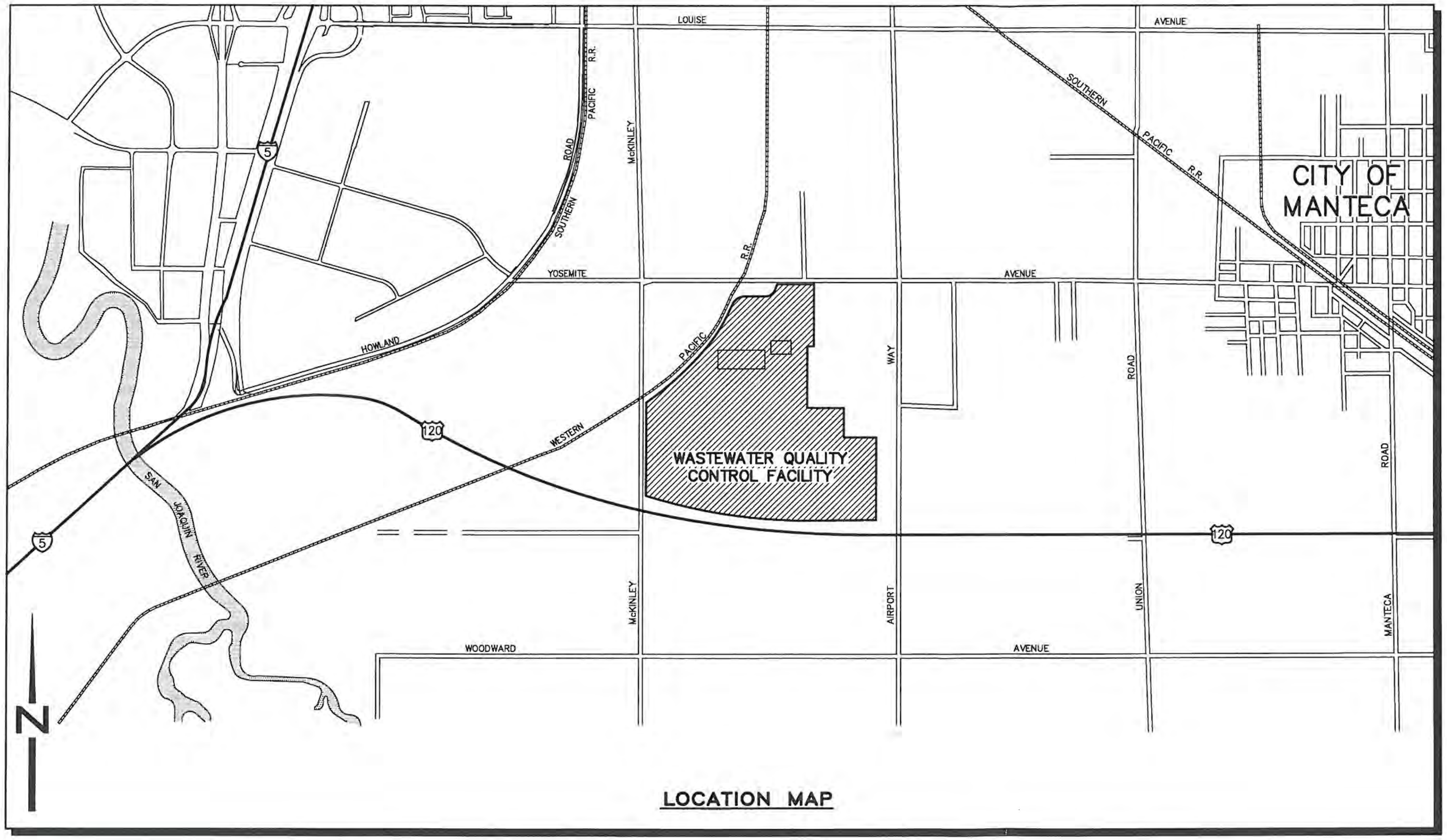
The Manteca WQCF is located on 210 acres of City owned property southwest of downtown Manteca at 2450 West Yosemite Avenue (see Figure 1-1). The WQCF treats typical municipal wastewater generated in the City of Manteca and the neighboring City of Lathrop. The plant also receives seasonal discharges from a local food processor (Eckert Cold Storage).

The Manteca WQCF began operation in 1959. At that time the facility consisted solely of an oxidation pond. Pond effluent was discharged to the surrounding land. In 1970, the first major upgrade to the plant occurred. This upgrade included the construction of preliminary and primary treatment facilities and aerobic sludge digestion. Effluent continued to be discharged to the land for agricultural applications. In 1986-1988, as part of the Clean Water Grant Program, a major expansion to the plant was constructed. This Phase I Expansion Project included the construction of secondary treatment facilities, anaerobic sludge digesters, sludge drying beds, a chlorine disinfection system, and an outfall to the San Joaquin River. Design capacity of the plant following the Phase I project was 5.45 mgd (ADWF). The Phase II Expansion Project in 1992-1993 added a primary sedimentation basin, secondary clarifier, and four sludge drying beds, increasing the facility capacity to 6.95 mgd (ADWF).

## 1.4 Description of Existing Facilities

Currently, the Manteca WQCF consists of an influent pump station with two mechanical screens, two aerated grit tanks, three primary sedimentation basins, a biotower feed pump station, two biotowers, five fine-bubble activated sludge aeration basins, three secondary clarifiers, a secondary effluent storage pond, and two chlorine contact tanks. Solid handling facilities include two dissolved air flotation units, two anaerobic digesters, one centrifugal dewatering system, and drying beds. Undisinfected secondary effluent is used to irrigate approximately 190 acres of City-owned land surrounding the plant. Flows in excess of crop demands are disinfected and discharged to the San Joaquin River.

Anaerobically digested sludge is dewatered, dried, stored on-site, and then transported to a local landfill. A site plan of the existing treatment plant can be seen in Figure 1-2.



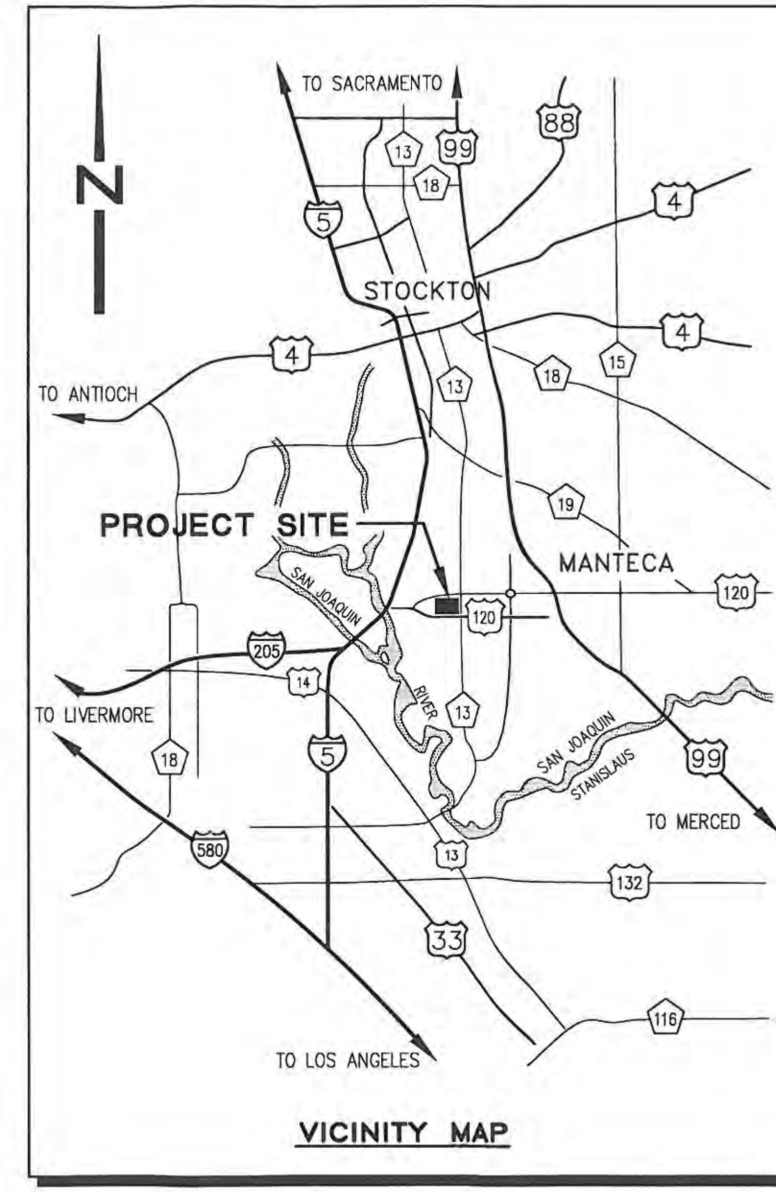




**FIGURE 1-1**

**MANTECA WASTEWATER QUALITY CONTROL FACILITY**
**MASTER PLAN UPDATE**

**LOCATION OF MANTECA WQCF**

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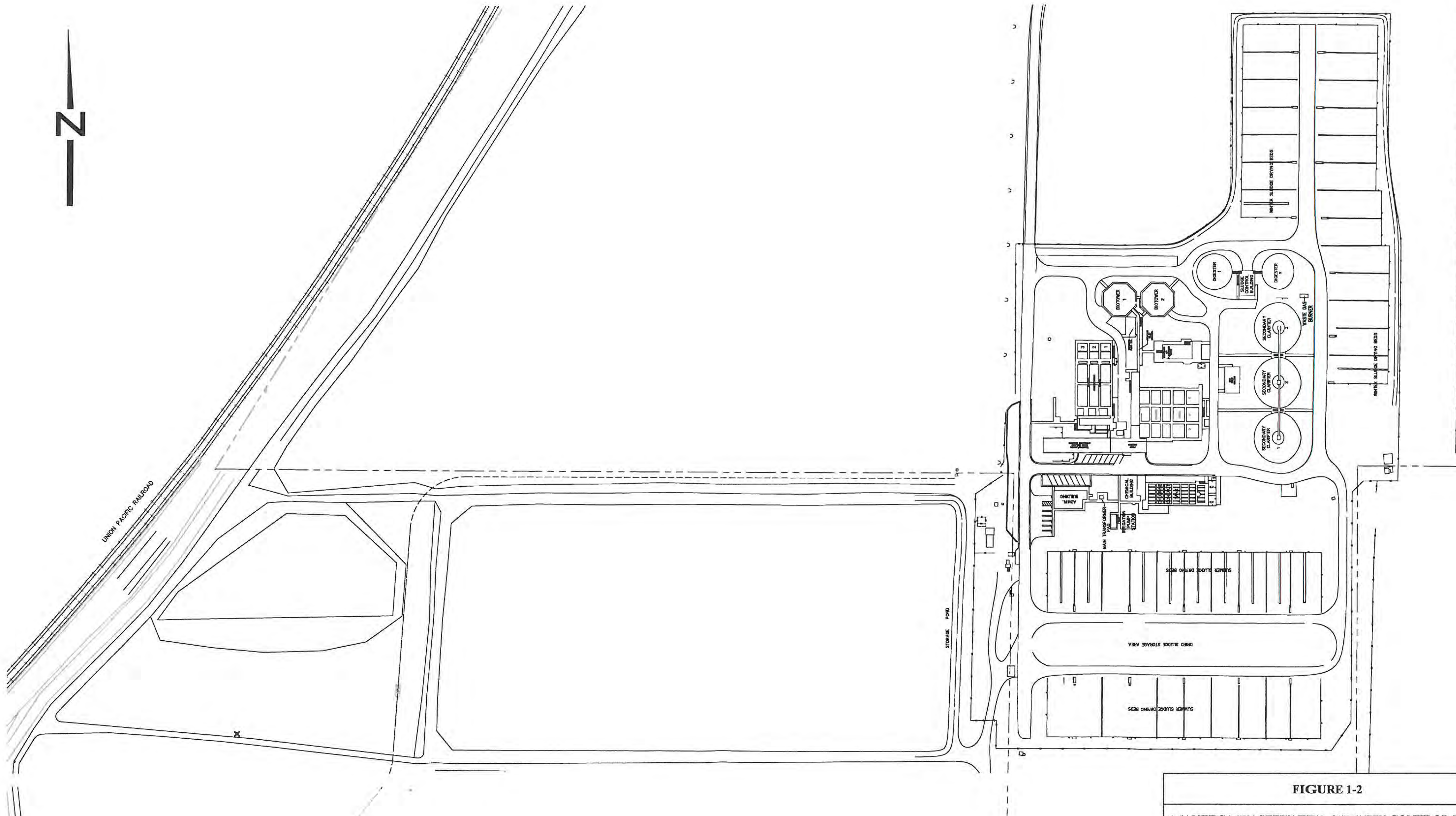


**FIGURE 1-2**

**MANTECA WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE**

PRIOR TO PHASE III EXPANSION PROJECT
SITE PLAN OF EXISTING TREATMENT PLANT

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## 1.5 Phase III Improvements

The Phase III Expansion Project is divided into four Schedules: A, B, C, and D, respectively (see Figure 1-3). Schedule A was completed in November 2003. Schedule B was substantially complete in November 2005. The anticipated dates for design completion, commencement of construction, and completion of construction for Schedules C and D are summarized in Table 1-1. A brief description of each of the four schedules is provided below.

**TABLE 1-1**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**PROBABLE SCHEDULE OF PHASE III IMPROVEMENTS**

| Project | Complete Design | Begin Construction | Complete Construction |
|---|---|---|---|
| Schedule C, Solids Handling | January 2007 | March 2007 | June 2008 |
| Schedule D, Effluent Filtration | Completed | January 2006 | June 2007 |

a. <u>Schedule A Improvements - Northside Nitrification Facilities</u>

Schedule A improvements included the construction of two new aeration basins, modifications to three existing secondary clarifier sludge collection mechanisms, and replacement of two existing centrifugal blowers with larger and more energy efficient units within the northside facilities (NSF). Schedule A improvements resulted in an increase in plant capacity from 6.95 to 7.5 mgd (ADWF). Improvements also resulted in the production of a nitrified effluent. Construction of Schedule A began in December 2001 and was recently completed. As part of the Schedule A improvements, the City also procured a skid-mounted centrifugal dewatering system to dewater anaerobically digested primary and secondary sludges to augment the existing drying bed facilities.

b. <u>Schedule B Improvements – Southside Facilities and Influent Pump Station</u>

Schedule B improvements included the construction of a new influent pump station, two aerated grit tanks, three primary sedimentation basins, five aeration basins, two secondary clarifiers, and an odor control biofilter. These primary and secondary treatment facilities are referred to as the southside facilities (SSF). Schedule B improvements also included the expansion of the existing laboratory and administration building. Schedule B improvements increased the treatment capacity of the plant from 7.5 mgd to 9.87 mgd (ADWF). Effluent, at the completion of Schedule B, will be nitrified and denitrified.

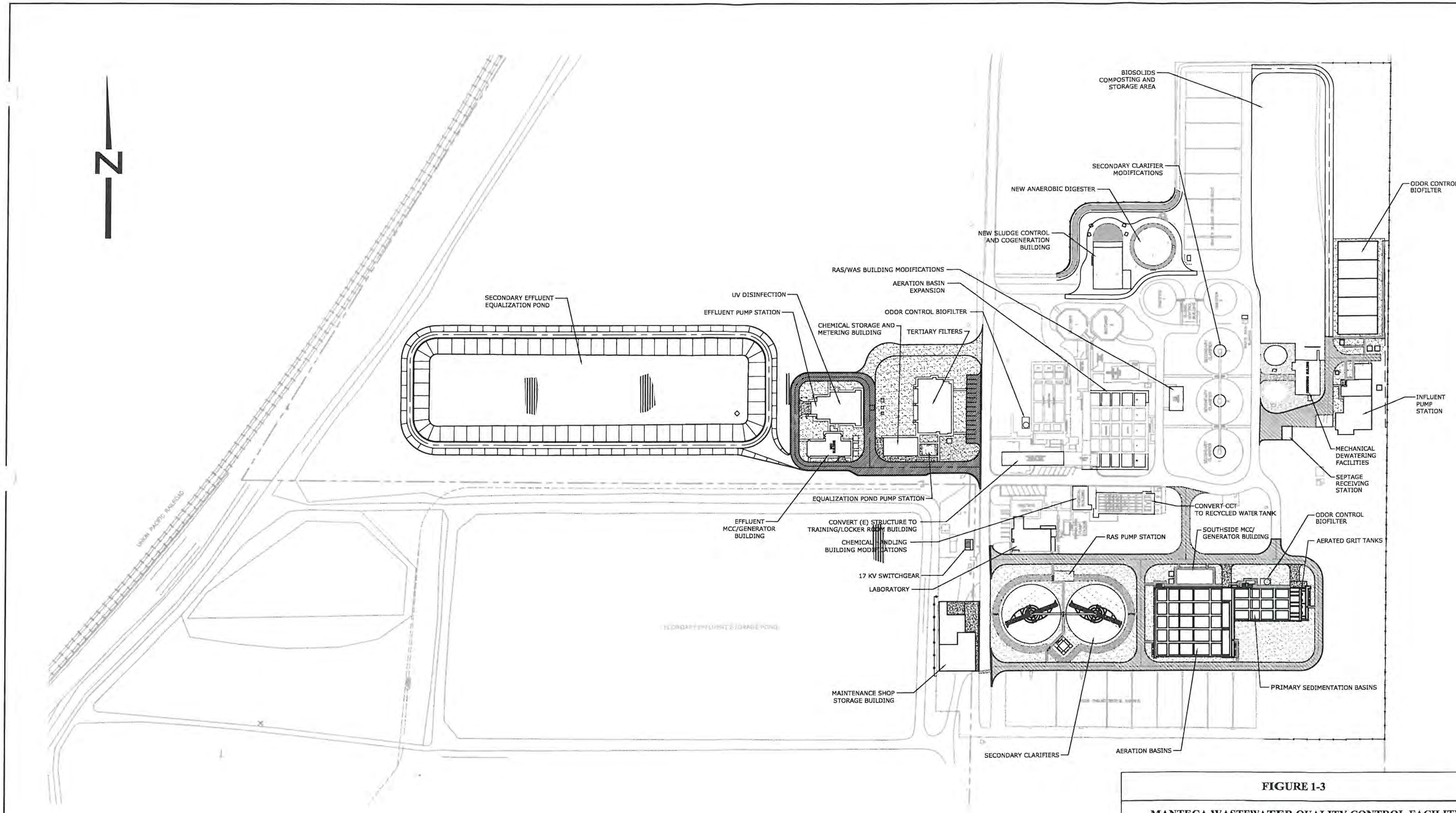


**FIGURE 1-3**

**MANTECA WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN UPDATE**

PHASE III EXPANSION PROJECT - SITE PLAN

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c. Schedule C Improvements – Solids Handling Facilities

The improvements proposed under Schedule C primarily involve solids handling. Under Schedule C, a mechanical dewatering facility will be constructed. The skid-mounted centrifugal dewatering system acquired under Schedule A will be relocated to a new dewatering building. Additional improvements to occur under Schedule C include the construction of a shop maintenance building and development of a paved biosolids stockpile area.

d. Schedule D Improvements – Tertiary Filtration and UV Disinfection Facilities

Proposed Schedule D improvements include the construction of a secondary effluent equalization pond, filter feed pump station, coagulation and flocculation facilities, tertiary filters, a chemical storage and handling facility, a UV disinfection system, an effluent pumping station, and two odor control biofilters. In addition to flow attenuation, the secondary effluent equalization pond will store water during periods of reverse river flow due to tidal influence in the San Joaquin River.

## 1.6 Industrial Pipeline Phase 3

Although not part of the Phase III Expansion Project, the City has initiated additional efforts to improve plant operations and effluent quality. Under the Industrial Pipeline Project Phase 3 (IPP-3), the existing secondary effluent storage pond will be modified to create a second lined pond for food processing wastewater from Eckert Cold Storage (Eckert). In the past, high-strength Eckert wastewater has been co-mingled with domestic wastewater and processed through the WQCF prior to disposal. IPP-3 is the final component in a source separation program designed to allow independent treatment of Eckert wastewater prior to land application at the WQCF. By segregating industrial wastewater, the operation of the plant facilities can be optimized with less risk of potential process upsets.

c. Schedule C Improvements – Solids Handling Facilities

The improvements proposed under Schedule C primarily involve solids handling. Under Schedule C, a mechanical dewatering facility will be constructed. The skid-mounted centrifugal dewatering system acquired under Schedule A will be relocated to a new dewatering building. Additional improvements to occur under Schedule C include the construction of a shop maintenance building and development of a paved biosolids stockpile area.

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# 2 Regulatory Setting

Future facility requirements are dictated by water quality regulations. A discussion of the regulatory setting for the Manteca WQCF is presented in this chapter.

## 2.1 Current Discharge Requirements

The Manteca WQCF operates under Regional Board Order No. R5-2004-0028 (NPDES Permit No. CA0081558) issued by the State of California Regional Quality Control Board (RWQCB) [2]. Water quality parameters for river discharge are summarized in Table 2-1. As seen in Table 2-1, beyond February 2009, water quality limitations will become more stringent for BOD, TSS, total coliform, and turbidity.

**TABLE 2-1**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**SUMMARY OF CURRENT WATER QUALITY REQUIREMENTS**
**FOR RIVER DISCHARGE**

| Constituent | Units | Requirement[a] (Effective through January 31, 2009) | Requirement (Effective as of February 1, 2009) |
|---|---|---|---|
| BOD | mg/L | 20 | 10 |
| TSS | mg/L | 20 | 10 |
| Total Coliform | MPN/100 mL | 23[b] | 2.2[b] |
| Turbidity | NTU | -- | 2[c] |
| Settleable Solids | mg/L | 0.1 | 0.1 |
| Chlorine Residual | mg/L | 0.01[db] | 0.01[d] |
| Oil and Grease | mg/l | 10 | 10 |
| Aluminum | µg/L | 71 | 71 |
| Conductivity | µmhos/cm | 1,000 | 1,000 |
| Ammonia (as N) | | | |
| June through September | mg/L | 2.1 | 2.1 |
| October through May | mg/L | 2.8 | 2.8 |
| Arsenic | µg/L | 10 | 10 |
| Copper | µg/L | 7.9 | 7.9 |
| Cyanide | µg/L | 3.7 | 3.7 |
| Iron | µg/L | 300 | 300 |
| Manganese | µg/L | 50 | 50 |
| Methylene Blue Active Substances (MBAS) | µg/L | 500 | 500 |
| Nitrate (as N) | mg/L | 10 | 10 |
| Nitrite (as N) | mg/L | 1 | 1 |

| Constituent | Units | Requirement[a] (Effective through January 31, 2009) | Requirement (Effective as of February 1, 2009) |
|---|---|---|---|
| Bis (2-ethylhexyl) phthalate | μg/L | 22 | 22 |
| Bromodichloromethane | μg/L | 5 | 5 |
| Dibromochloromethane | μg/L | 1.4 | 1.4 |
| Mercury | lb/yr | 0.69[e] | 0.69[e] |
| 2,4,6-Trichlorophenol | μg/L | 34 | 34 |

[a] Monthly average, unless noted otherwise
[b] Weekly median
[c] 24-hour average
[d] 4 day average
[e] Corresponds to 0.028 μg/L for the average treatment design capacity of 8.11 mgd

## 2.2 Current Effluent Quality

Typical effluent quality (for conventional parameters) prior to the Phase III Expansion Project is presented in Table 2-2. The results of effluent sampling during 2002 for additional constituents are summarized in Table 2-3.

**TABLE 2-2**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**TYPICAL EFFLUENT QUALITY PRIOR TO PHASE III EXPANSION PROJECT**

| Constituent | Units | Effluent Quality 30-day Average |
|---|---|---|
| $BOD_5$ | mg/L | 16 |
| Settleable Solids | mg/L | 0.1 |
| Total Suspended Solids | mg/L | 19 |
| Total Coliform Organisms | MPN/100 mL | 23[a] |
| Oil and Grease | mg/L | 2.1 |
| Chlorine Residual | mg/L | -- |
| Ammonia | mg/L | 20 |

[a] Weekly median value

**TABLE 2-3**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**SUMMARY OF EFFLUENT QUALITY SAMPLING**
**FOR ADDITIONAL CONSTITUENTS[a]**
**PRIOR TO PHASE III EXPANSION PROJECT**

| Constituent | Units | Range | Mean | Maximum |
|---|---|---|---|---|
| Aluminum | µg/L | 70 - 350 | 150 | 350 |
| Conductivity | µmhos/cm | 819 - 1300 | 1099 | 1300 |
| Arsenic | µg/L | 11 - 14 | 12.5 | 14 |
| Copper | µg/L | 6.8 - 13 | 9 | 13 |
| Cyanide | µg/L | 6 - 31 | 7 | 31 |
| MBAS[b] | µg/L | 120 - 1800 | 620 | 1800 |
| Nitrate (as N) | mg/L | 1 - 7.1 | - | - |
| Nitrite (as N) | mg/L | 0.12 - 1.80 | - | - |
| Bis (2-ethylhexyl) phthalate | µg/L | 0.9 - 7 | 3.48 | 7 |
| Bromodichloromethane | µg/L | 1 - 3.5 | 1.98 | 3.5 |
| Dibromochloromethane | µg/L | ND[c] - 1.2 | 0.47 | 1.2 |
| Mercury | µg/L | 0.013 - 0.028 | - | - |
| 2,4,6-Trichlorophenol | µg/L | 0 - 11 | 3.28 | 11 |

[a] Based on 2002 monitoring results for all constituents except conductivity, MBAS, nitrate, and nitrite. Monitoring results for MBAS, conductivity, nitrate, and nitrite extend from 1998-2002.
[b] Methylene Blue Active Substances.
[c] Non-detect.

## 2.3 Probable Future Effluent Quality

An analysis of probable effluent quality following the completion of proposed improvements scheduled as part of the Phase III Expansion Project is presented below.

Current plant effluent quality is expected to improve significantly after the completion of Phase III modifications and expansion. The removal of ammonia and total nitrogen are the major effluent quality enhancements anticipated under Schedule A and Schedule B of the Phase III Project, respectively. Phase III Schedule D improvements will further improve the effluent quality to meet Title 22 wastewater reuse criteria for total suspended solids, turbidity, BOD, and total coliforms. Effluent quality values for conventional parameters, which served as design objectives for the Phase III Expansion project, are presented in Table 2-4. To determine removal efficiencies for certain constituents following completion of the proposed modifications and expansions, reported removal efficiencies at similar facilities in Northern California were investigated to bracket probable process performance.

**TABLE 2-4**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**PROJECTED EFFLUENT QUALITY AFTER PHASE III EXPANSION[a]**

| Constituents | Units | Schedule A | Schedule B | Schedule D |
|---|---|---|---|---|
| BOD | mg/L | 15 | 10 | 7 |
| Total Suspended Solids | mg/L | 15 | 10 | < 10 |
| Total Coliform | MPN/100 mL | 23 | 23 | < 2.2 |
| Turbidity | NTU | 8 | 8 | < 2 |
| Settleable Solids | mg/L | 0.1 | 0.1 | 0.1 |
| Chlorine Residual | mg/L | 0.01 | 0.01 | 0.01 |
| Oil and Grease | mg/L | 2.1 | 2.1 | 2.1 |
| Aluminum | µg/L | 150 | 150 | 150 |
| Electrical Conductivity | µmhos/cm | 1100 | 1100 | 825 |
| Ammonia | mg/L | 2.1 | 1.5 | 1.5 |
| Arsenic | µg/L | 12.5 | 12.5 | 8 |
| Copper | µg/L | 9 | 9 | 7 |
| Cyanide | µg/L | 7 | 7 | 1 |
| Iron[b] | µg/L | 47 | 47 | 47 |
| Manganese[b] | µg/L | 15 | 15 | 15 |
| MBAS | µg/L | 160 | 160 | 160 |
| Nitrate (as N) | mg/L | 10 | 5 | 5 |
| Nitrite (as N) | mg/L | 1 | 1 | 1 |
| Bis (2-ethylhexyl) phthalate | µg/L | 3.48 | 3.48 | 3.48 |
| Bromodichloromethane | µg/L | 1.98 | 1.98 | ND[c] |
| Dibromochloromethane | µg/L | 0.47 | 0.47 | ND[c] |
| Mercury | µg/L | 0.028 | 0.028 | 0.010 |
| 2,4,6-Trichlorophenol | µg/L | 3.28 | 3.28 | 3.28 |

[a] Projected mean values.
[b] Concentration represents median value taken from analysis at WQCF effluent evaluated as part of WQCF Special Studies Monitoring Program
[c] Non-detect.

The projected effluent quality following each stage of construction is presented in Table 2-4. Projected effluent concentrations reflect baseline values established for secondary treatment at the WQCF and anticipated facility enhancements. These concentrations were developed considering the following:

1. Historical WQCF secondary effluent values prior to Phase III Schedule A improvements. For planning purposes, average (mean) and maximum measured values were used to determine effluent quality.

2. Anticipated improvements in potable water quality due to groundwater treatment and introduction of a surface water supply source.

3. Typical removal efficiencies reported in the literature for similar treatment facilities including a survey of six San Francisco Bay area treatment plants (Petaluma, Palo Alto, Half Moon Bay, Napa, Calistoga, San Jose-Santa Clara).

4. Results of special studies undertaken at WQCF effluent.

A discussion of specific effluent quality parameters follows below. The discussion is organized into three areas: a) Reduction in constituents due to treatment process improvements; b) improvements in water quality attributable to source water enhancements; and c) constituents not impacted by the proposed facility configuration following the Phase III Expansion.

a. Reduction in Constituents due to Treatment Process Improvements

The upgrade of the activated sludge process and the addition of tertiary filtration will significantly impact conventional pollutant concentrations. These are discussed as follows.

**BOD, Suspended Solids, Coliform, Organics:** With a high-rate activated sludge process, an easily filterable secondary effluent will be produced. Following tertiary filtration in accordance with Title 22 standards and UV disinfection conforming to National Water Research Institute (NWRI) guidelines, BOD and suspended solids concentrations will be less than 10 mg/L consistently (approximately 7 mg/L), turbidities will be less than 2 NTU, while total coliform levels will be less than 2.2 MPN/100 mL. In terms of organics and mercury, based on historical operating experience at other facilities, unless chlorination is practiced, effluent concentrations of bromodichloromethane and dibromochloromethane should be at the non-detect level while mercury levels should be less than 0.010 μg/L.

**Nitrogen:** A high level of nitrogen removal will be accomplished through the activated sludge nitrification – denitrification process. Ammonia will be largely converted to nitrate and nitrite. Denitrification facilities will then essentially complete the nitrogen removal process through the conversion of nitrates and nitrites to nitrogen gas.

**Copper:** The mean and maximum effluent values observed for copper prior to the Phase III Expansion were 9 μg/L and 13 μg/L respectively. Historical removal efficiencies at similar advanced treatment plants are on the order of 25 percent. Considering 25 percent removal through advanced wastewater treatment (filtration), mean and maximum effluent concentrations of 7 μg/L and 10 μg/L are anticipated for copper.

**Cyanide:** The mean and maximum effluent values observed for cyanide prior to the Phase III expansion were 7 μg/L and 31 μg/L. Removal of cyanide with filtration is insignificant. Some coincidental removal may occur but is likely in the 5 - 10 percent range. Cyanide concentrations will likely be reduced, however, with UV disinfection. Cyanide has not been detected at any of the city wells, and there are no known industrial dischargers. Because source water changes will not impact cyanide levels and removal efficiencies through UV disinfection are significant, a maximum effluent concentration of 1 μg/L is predicted.

**Methylene Blue Active Substances (MBAS):** The mean and maximum effluent values observed for MBAS prior to the Phase III expansion were 620 μg/L and 1,800 μg/L. MBAS is removed through the biological treatment processes. The upgrade of the activated sludge process to full nitrification (Schedule A) will increase the removal efficiency for MBAS. Based on 75 percent removal, the MBAS concentration is expected to decrease to approximately 450 μg/L (maximum). The anticipated future mean concentration is 160 μg/L following completion of Phase III improvements.

b. Improvements in Water Quality Attributable to Source Water Enhancements

The City of Manteca is implementing two major programs to enhance source water quality: arsenic treatment of groundwater and participation in a surface water supply project. Specific reductions in effluent concentrations due to source water changes are described below.

**Arsenic:** The maximum effluent value observed for arsenic prior to the Phase III expansion was 14 μg/L. Arsenic in the WQCF effluent originates from groundwater with concentrations ranging from 5 - 20 μg/L. Arsenic is not generated during the treatment process at the WQCF. Treatment will be implemented, however, at City wells to reduce arsenic concentrations to 8 μg/L. Based on the source water improvements, an average effluent concentration of 8 μg/L at the WQCF may be achievable if the water treatment target is achieved and no other arsenic is added as a compound of wastewater.

**Electrical Conductivity:** The WQCF effluent value for electrical conductivity ranged from 819 - 1300 μmhos/cm during 2003. Electrical conductivity is largely a function of source water quality and mineral pick-up during water use. The relatively high value of electroconductivity (and corresponding salinity) is due to the exclusive use of groundwater in the City. Because chemical addition at the WQCF is seldom practiced, additional increases in electroconductivity at the plant are unlikely.

In the future, surface water from the South County Water Supply Program will be introduced and blended with groundwater for the City water supply. The conductivity of surface water is approximately 100 μmhos/cm. Considering the likely blend of surface water and groundwater for the potable water supply, electrical conductivity is expected to decrease up to 25 percent leading to a similar decrease in plant effluent conductivity. These conductivity levels in the effluent are not certain until the actual water supply conditions have stabilized.

c. Constituents Not Impacted by Proposed Facility Configuration

A number of dissolved constituents will largely pass through the treatment plant and not be removed appreciably as part of the facility improvements/reconfiguration. These include aluminum, iron, bis (2-ethylhexyl) phthalate and 2,4,6-trichlorophenol. Because reductions in concentrations are unlikely, predicted future values represent the median of historical measured values.

## 2.4 Probable Future Water Quality Limitations

A summary of probable future water quality requirements beyond Year 2009 for continued discharge to the San Joaquin River is presented below. Probable future water quality requirements are based on information in the current NPDES permit. The RWQCB considers the new permit to include the most stringent water quality criteria foreseeable for the WQCF discharge.

A discussion of each of the water quality parameters is presented along with the future probable permit level considering the current NPDES permit, relevant water quality criteria, and the results of site specific studies. Probable future water quality requirements along with the relevant water quality criteria are summarized in Table 2-5.

**TABLE 2-5**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**SUMMARY OF PROBABLE FUTURE WATER QUALITY REQUIREMENTS FOR RIVER DISCHARGE**

| Constituent[a] | Units | Probable Future Requirement (monthly average unless noted otherwise) | Relevant Most Stringent Water Quality Criteria |
|---|---|---|---|
| BOD | mg/L | 10 or lower | -- |
| TSS | mg/L | 10 or lower | -- |
| Total Coliform | MPN/100 mL | 2.2[b] | 2.2 |
| Turbidity | NTU | 2[c] | 2 |
| Settleable Solids | mg/L | 0.1 | -- |
| Chlorine Residual | mg/L | 0.01[d] | 0.011 |
| Oil and Grease | mg/L | 10 | -- |
| Aluminum | µg/L | 71 | 87 |
| Conductivity | µmhos/cm | 1,000 | 1,000 |
| Ammonia (as N) | | | |
| June through September | mg/L | 2.1 or lower | 0.62 |
| October through May | mg/L | 2.8 | 5.62 |
| Arsenic | µg/L | 10 | 10 |
| Copper | µg/L | 7.9 | 10 |
| Cyanide | µg/L | 3.7 | 10 |
| Iron | µg/L | 300 | 300 |
| Manganese | µg/L | 50 | 50 |
| MBAS | µg/L | 500 | 500 |
| Nitrate (as N) | mg/L | 10 | 10 |

| Constituent[a] | Units | Probable Future Requirement (monthly average unless noted otherwise) | Relevant Most Stringent Water Quality Criteria |
|---|---|---|---|
| Nitrite (as N) | mg/L | 1 | 1 |
| Bis (2-ethylhexyl) phthalate | μg/L | 22 | 1.8 |
| Bromodichloromethane | μg/L | 5 | 0.56 |
| Dibromochloromethane | μg/L | 1.4 | 0.41 |
| Mercury | -- | 0.69 lb/yr or lower[e] | 0.05 μg/L |
| 2,4,6-Trichlorophenol | μg/L | 34 | 2.1 |

[a] For constituents listed under the current NPDES permit
[b] Weekly median
[c] 24 hour average using 1 hour average values
[d] 4-day average
[e] Corresponds to 0.023 μg/L for the average design treatment capacity of 9.87 mgd

a. Biochemical Oxygen Demand (BOD)

The Basin Plan (BP) prescribes a minimum dissolved oxygen (DO) concentration in the San Joaquin River of 5 mg/L [3]. This standard is applicable throughout the year. The Bay/Delta plan prescribes a minimum DO concentration of 6.0 mg/L in the San Joaquin River within the reach from Turner Cut to Stockton from September through November. The DO objectives are not met frequently in the San Joaquin River, leading to the Clean Water Act section 303(d) listing. A total maximum daily load (TMDL) implementation plan was submitted to the RWQCB in February 2003. The TMDL was developed and submitted to the EPA in June 2003. Further action by the City to reduce its impact on the San Joaquin River DO concentration, beyond the BOD requirements of the new permit will not be required by the RWQCB until the TMDL for DO has been approved by EPA. The new NPDES order contains a provision to allow for modifications to the BOD effluent limitation after the DO TMDL is finalized. As such, future BOD effluent requirement may be reduced below 10 mg/L based on the results of the DO TMDL study.

b. Total Suspended Solids (TSS)

The DO TMDL discussed above will also be used to evaluate the future requirements for TSS. The new NPDES order contains a provision to allow for modifications for the TSS effluent limitation after the DO TMDL is finalized. Similarly, future TSS effluent requirement may be reduced below 10 mg/L based on the results of the DO TMDL study.

c. Total Coliform

The total coliform requirements in the new NPDES permit are expected to remain unchanged in the future.

d. Turbidity

The new NPDES permit requires the daily average effluent turbidity to be less than 2 NTU. An effluent turbidity of 2 NTU is equivalent to approximately 3-5 mg/L TSS, which is half of the permitted TSS requirement. Considering TSS requirements, the turbidity values in the new NPDES permit are assumed to remain unchanged in the future.

e. Settleable Solids

The settleable solids requirements in the new NPDES permit are expected to remain unchanged in the future.

f. Chlorine Residual

The BP contains a narrative toxicity objective. EPA ambient water quality criteria for chlorine (for protection of fresh aquatic life) were used in the new NPDES permit to implement the narrative toxicity objective. Chlorine requirements specified in the new NPDES permit are expected to remain unchanged in future permits assuming EPA water ambient water quality criteria for chlorine is not modified in the future.

g. Oil and Grease

The oil and grease requirements in the new NPDES permit are expected to remain unchanged in the future.

h. Aluminum

The BP contains a narrative toxicity objective, and chemical constituents water quality objective prohibits chemical constituents in concentrations that exceed state maximum contaminant levels (MCLs) or that adversely affect beneficial uses. MUN is a beneficial use of the San Joaquin river. To implement the narrative toxicity objective, the EPA ambient water quality criteria for aluminum (for protection of fresh aquatic life) was used to establish the requirement for aluminum in the new NPDES permit. No dilution credit was given in the development of the aluminum limitation, therefore limits specified in the new NPDES permit are not expected to change significantly in the future assuming EPA water ambient water quality criteria for aluminum is not modified.

i. Electrical Conductivity

The BP chemical constituent water quality objective incorporates state MCLs, and contains numeric water quality objectives for electrical conductivity (EC). Based on the results of the TMDL study for salinity and boron in lower San Joaquin river, the RWQCB concluded that the receiving water frequently has no assimilative capacity for EC. The EC effluent limitations in the new NPDES permit are based on the BP water quality objective in the South Delta.

Future NPDES EC limitations are not expected to change significantly assuming BP chemical water quality objectives are not modified.

j. Ammonia

Ammonia effluent limitations included in the new NPDES permit are based on the BP narrative toxicity objective and the EPA ambient water quality criteria (for the protection of freshwater aquatic life) for ammonia. Ammonia limitations in the new NPDES permit from October through May are based on acute levels which are more stringent than chronic levels during cold weather conditions. Ammonia limitations from June through September are based on chronic levels which are more stringent than acute levels during warm weather.

No dilution credit was given in the NPDES effluent ammonia limitation calculations during cold weather period, therefore future limitations from October through May are not expected to change significantly assuming BP objectives and the EPA ambient water quality criteria for ammonia is not modified. A dilution credit of four to one was used in effluent ammonia limitation calculations during the warm weather period in the permit, therefore a more stringent ammonia limit for the period from June through May may be adopted if a lower dilution factor is applied in the future.

k. Arsenic

The arsenic effluent limitation included in the new NPDES permit is based on the BP numeric objective for San Joaquin-Sacramento Delta. The BP objective and EPA primary MCL are more stringent than the California Toxic Rule (CTR) criteria for arsenic. Arsenic limitations in the NPDES are calculated based on acute levels which are also more stringent than chronic levels. No dilution credit was given in the new NPDES effluent limitation calculation for arsenic, therefore future limitations are not expected to change significantly assuming the BP objective and arsenic primary MCL are not modified.

l. Copper

The copper effluent limitation included in the new NPDES permit is based on the BP objectives for the protection of fresh water species. The BP objective is more stringent than the CTR criteria for copper. Copper limitations in the NPDES are calculated based on acute levels which are more stringent than chronic levels. No dilution credit was given and the default EPA translator was used in the NPDES effluent limitation calculation for copper, therefore future limitations are not expected to change significantly assuming the BP objective is not modified.

m. Cyanide

The cyanide effluent limitation included in the new NPDES permit is based on the BP objective. The BP objective is more stringent than the CTR criteria for cyanide. The cyanide limitation is calculated based on acute levels which are more stringent than chronic levels. No dilution credit was given in the NPDES effluent limitation calculation for cyanide; therefore, the future cyanide limitation is not expected to change significantly assuming the BP objective is not modified.

n. Iron

The BP chemical constituent objective includes a receiving water objective for iron in the Delta and a secondary MCL for iron in drinking water. The iron effluent limitation included in the new NPDES permit is based on the BP chemical constituents objective for iron. No dilution credit was given in the NPDES effluent limitation calculation for iron, therefore the future iron limitation is not expected to change significantly assuming the BP chemical constituents objective is not modified.

o. Manganese

The BP chemical constituent objective includes a receiving water objective for manganese in the Delta, and a secondary MCL for manganese in drinking water. The manganese effluent limitation included in the NPDES permit is based on the BP chemical constituents objective for manganese. No dilution credit was given in the NPDES effluent limitation calculation for manganese, therefore the future manganese limitation is not expected to change significantly assuming the BP chemical constituents objective is not modified.

p. Methylene Blue Active Substances (MBAS)

The BP includes the chemical constituents objective that incorporates state MCLs for waters designated as MUN (Municipal). MBAS concentrations in excess of the secondary MCL-consumer acceptance limit produce aesthetically undesirable froth, taste, and odor. The MBAS effluent limitation included in the NPDES permit is based on the BP water quality objectives for chemical constituents, floating material, and taste/odors. No dilution credit was given in the NPDES effluent limitation calculation for MBAS, therefore the future limitation is not expected to change significantly assuming the BP chemical water quality objectives are not modified.

q. Nitrate and Nitrite (as N)

The BP chemical constituents water quality objective prohibits chemical constituents in concentrations that exceed drinking water MCLs published in Title 22 of the California Code of regulations or that adversely affect beneficial uses. Municipal and domestic water supplies are beneficial uses of the San Joaquin river. The California Department of Health Services (DHS) has adopted primary MCLs for the protection of human health for nitrite and nitrate. NPDES effluent limitations for nitrite and nitrate are based on the primary MCLs. Future nitrite and nitrate limitations are not expected to change significantly assuming primary MCLs for nitrite and nitrate are not modified.

r. Bis(2-ethylhexyl)phthalate

The CTR includes standards for the protection of human health based on a one in a million cancer risk for carcinogenic constituents like Bis(2-ethylhexyl)phthalate. The effluent limitation in the NPDES for Bis(2-ethylhexyl)phthalate is based on the Bis(2-ethylhexyl)phthalate CTR criteria for the protection of human health. The human health based criteria are based on lifetime exposures. The effluent limitation calculation procedures in the State Implementation Policy (SIP) allow for granting a dilution credit. However, the RWQCB is not required to grant a mixing zone or allocate the full assimilative capacity of the receiving water. Bis(2-ethylhexyl)phthalate was not detected in the receiving water. A steady state analysis utilizing the harmonic mean flow of San Joaquin River at Vernalis and the permitted discharge flow (based on Phase III expansion) provides a dilution ratio of 162:1. For limitations based on human health criteria, the dilution credit is limited to the amount required to maintain compliance. The dilution credit required to maintain compliance for Bis(2-ethylhexyl)phthalate was established as 13.3 in the new NPDES permit.

The dilution ratio of 162:1 (based on the Phase III Expansion flow) is estimated to decrease to approximately 60:1 at an increased WQCF discharge of 27 mgd in the future. Because the required dilution credit of 13.3 is significantly lower compared to the estimated future dilution ratio of 60:1, future limitation for Bis(2-ethylhexyl)phthalate is not expected to change significantly assuming the CTR criteria for Bis(2-ethylhexyl)phthalate is not modified.

s. Bromodichloromethane (BDCM)

The CTR includes standards for the protection of human health based on a one in a million cancer risk for carcinogenic constituents like BDCM. The effluent limitation in the NPDES for BDCM is based on the BDCM CTR criteria for the protection of human health. BDCM was not detected in the receiving water. The dilution credit required to maintain compliance for BDCM was established as 11.5 in the new NPDES permit.

As discussed previously, the dilution ratio is projected to decrease to approximately 60:1 in the future. Because the dilution credit of 11.5 is significantly lower than the estimated future dilution ratio of 60:1, the future BDCM limitation is not expected to change significantly assuming the CTR criteria for BDCM is not modified.

t. Dibromochloromethane (DBCM)

The CTR includes standards for the protection of human health based on a one in a million cancer risk for carcinogenic constituents like DBCM. The effluent DBCM limitation in the new NPDES is based on the DBCM CTR criteria for the protection of human health. Dibromochloromethane was not detected in the receiving water. The dilution credit required to maintain compliance for DBCM was established as 8.9 in the new NPDES permit. As noted earlier, the dilution ratio is projected to decrease to approximately 60:1 in the future. Because the required dilution credit of 8.9 is significantly lower than the estimated future dilution ratio of 60:1, the future DBCM limitation is not expected to change significantly assuming the DBCM CTR criteria is not modified.

u. Mercury

The mercury limitation in the new NPDES permit was established based on the BP narrative toxicity objective and the issues related to bioaccumulation of mercury in fish tissue. There is currently no assimilative capacity for mercury in the San Joaquin River. A TMDL for mercury is under development. The interim mercury limitation in the NPDES permit for mercury is based on the maximum mercury concentration observed in the WQCF effluent and resultant annual mass loading from the maximum concentration. A lower mercury limitations may be adopted upon the development of the TMDL, if the RWQCB determines that a mercury offset program is feasible for dischargers subject to a NPDES permit.

Even though the WQCF effluent does not contain significant levels of mercury (the maximum observed WQCF effluent mercury concentration of 0.028 μg/L is less than the water and organism criteria of 0.05 μg/L), future NPDES permits may have more stringent mercury limitations because of the following: 1) there is currently no assimilative capacity for mercury in the San Joaquin River; 2) the receiving water has much lower mercury levels (0.0036-0.0093 μg/L in receiving water versus 0.013-0.028 μg/L in WQCF effluent); and, 3) the TMDL study may impose lowered mercury load allocations for the WQCF.

v. 2,4,6-Trichlorophenol

The CTR includes standards for the protection of human health based on a one in a million cancer risk for carcinogenic constituents like 2,4,6-Trichlorophenol. The effluent limitation in the NPDES permit for 2,4,6-Trichlorophenol is based on the 2,4,6-Trichlorophenol CTR criteria for the protection of human health. 2,4,6-Trichlorophenol was not detected in the receiving water. The dilution credit required to maintain compliance for 2,4,6-Trichlorophenol was established as 16.9 in the new NPDES permit.

As discussed previously, the dilution ratio is estimated to decrease to approximately 60:1 in the future. Because the required dilution credit of 16.9 is significantly lower than the estimated future dilution ratio of 60:1, future 2,4,6-Trichlorophenol limitation is not expected to change significantly assuming the CTR criteria for 2,4,6-Trichlorophenol is not modified.

w. Other Water Quality Parameters

Although limits were not imposed in the new NPDES permit, a number of additional water quality parameters were addressed. Because these parameters may be included in future permits, each is discussed below.

**Molybdenum:** Molybdenum was not monitored in the plant effluent or in the receiving water. A molybdenum limitation may be included in the new and/or future permits, if future monitoring shows a reasonable potential to deteriorate a water quality objective. The likely limitation would be 10 μg/L (without any dilution credit), which is the recommended agricultural water quality goal for molybdenum to implement the BP narrative chemical constituent objective.

**Carbofuran:** Carbofuran was detected in the effluent and receiving water, but exact levels were not quantified due to the laboratory's reporting limit for carbofuran. If future monitoring shows a reasonable potential to deteriorate a water quality objective, a carbofuran limitation may be included in the new and/or future permits. The likely limitation will be 18 μg/L (without any dilution credit), which is the MCL criteria for carbofuran. The State of California Office of Environmental Health Hazard Assessment (OEHHA) criterion for carbofuran, which is 1.7 μg/L, may also be adopted instead of the carbofuran MCL.

**Zinc:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality objective by exceeding the CTR MCL for zinc. Zinc may be included in future NPDES permits because the maximum observed concentration of 42 μg/L is close to the current criteria (continuous maximum criteria is 48 μg/L and the BP objective is 100 μg/L).

**Lindane:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria for lindane. Lindane was not detected in the WQCF effluent (at a reporting limit of 0.01 μg/L). Lindane should not be present in detectable concentrations according to the BP. The acceptable detection limit for Lindane is 0.02 μg/L. Lindane may be included in future NPDES permits if concentrations above accepted detection limits are observed in the future.

**Endrin Aldehyde:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria for Endrin aldehyde. Endrin aldehyde was not detected in the WQCF effluent (at a detection level of 0.01 μg/L). Endrin aldehyde should not be present in detectable concentrations according to the BP. The acceptable detection limit for Endrin aldehyde is 0.01 μg/L. Endrin aldehyde may be included in future NPDES permits if concentrations above accepted detection limits are observed in the future.

**4,4'-DDT:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria for 4,4'-DDT. 4,4'-DDT was not detected in the WQCF effluent (at a reporting level of 0.01 μg/L). 4,4'-DDT should not be present in detectable concentrations according to the BP. The acceptable detection limit for 4,4'-DDT is 0.01 μg/L. 4,4'-DDT may be included in future NPDES permits if concentrations above accepted detection limits are observed in the future.

**PCBs:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria for PCBs. PCBs were not detected in the

WQCF effluent at a reporting level of 0.1 μg/L. However, the reporting limit for PCBs was not low enough to compare with the relevant CTR criteria of 0.00017 μg/L. PCBs will be monitored at lower detection limits and may be included in future NPDES permits if concentrations above the CTR criteria are observed .

**2,3,7,8-TCDD (Dioxin):** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria for dioxin. Dioxin was not detected in the WQCF effluent at a reporting level of 0.975 pg/L. However, the reporting limit for dioxin is not low enough to compare with the relevant CTR criteria of 0.013 pg/L. Dioxin will be monitored at lower detection limits and may be included in future NPDES permits if concentrations above the CTR criteria are observed .

**Total Trihalomethanes and Chloroform:** The discharge does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality objective for MUN use by exceeding the EPA primary MCL for TTHMs or the OEHHA criteria for chloroform. TTHMs and chloroform will decrease in the future (reducing the reasonable potential levels) when the chlorination system is replaced by UV disinfection.

**Chloromethane:** The discharge does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria. One possible source of chloromethane is chlorination. Lower levels of chloromethane may be observed in the future (reducing the reasonable potential levels) when the chlorination system is replaced by UV disinfection.

**Dichloromethane:** The discharge does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria. One possible source of dichloromethane is chlorination. Lower levels of dichloromethane may be observed in the future (reducing the reasonable potential levels) when the chlorination system is replaced by UV disinfection.

**Chromium (VI):** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality objective by exceeding the CTR MCL for chrominum (VI). Chrominum (VI) is unlikely to be included in future NPDES permits if significant deviations from current conditions do not occur. The maximum effluent concentration of 0.6 μg/L is significantly lower than the current criteria (continuous maximum criteria is 16 μg/L and the secondary MCL is 50 μg/L).

**Silver:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality objective by exceeding the CTR MCL for silver. Silver is unlikely to be included in future NPDES permits if significant deviations from current conditions do not occur. The maximum effluent concentration of 3.2 μg/L is significantly lower than the current criteria (continuous maximum criteria is 8.6 μg/L and the BP objective is 10 μg/L).

**Lead:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality objective by exceeding the CTR MCL for lead. Lead is unlikely to be included in future NPDES permits if significant deviations from current conditions do not occur. The maximum effluent concentration of 1.3 μg/L is significantly lower than the current criteria (secondary MCL is 15 μg/L and the continuous maximum criteria is 150 μg/L).

**1,4 Dichlorobenzene:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria. 1,4-Dihlorobenzene is unlikely to be included in future NPDES permits if significant deviations from current conditions do not occur. The maximum effluent concentration of 0.8 μg/L is significantly lower than the current criteria (the secondary MCL is 5 μg/L).

**Toluene:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria for toulene. Toluene is unlikely to be included in future NPDES permits if significant deviations from current conditions do not occur. The maximum effluent concentration of 0.7 μg/L is significantly lower than the current criteria (the secondary MCL is 150 μg/L).

**MTBE:** The discharge currently does not have a reasonable potential to cause or contribute to an in-stream excursion above the water quality criteria for MTBE. MTBE is unlikely to be included in future NPDES permits if significant deviations from current conditions do not occur. The maximum effluent concentration of 0.7 μg/L is significantly lower than the current criteria (the secondary MCL is 5 μg/L).

## 2.5 Comparison of Projected Effluent Quality With Probable Future Quality Limitations

Table 2-6 compares the projected effluent quality from the WQCF following completion of the Phase III Expansion Project to the probable future water quality limitations requirements. A discussion of potential compliance issues follows below. Compliance with limits considers three categories: 1) limits that can be achieved; 2) limits that may be achieved; and 3) limits that are unlikely to be achieved.

**TABLE 2-6**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**COMPARISON OF PROBABLE FUTURE WATER QUALITY LIMITATIONS**
**VERSUS PROJECTED EFFLUENT QUALITY**
**FOLLOWING PHASE III EXPANSION PROJECT**

| Constituents | Units | Compliance Criteria | Probable Future Limit | Projected Effluent Median Concentration |
|---|---|---|---|---|
| BOD | mg/L | Monthly average | 10 or lower | 7 |
| Total Suspended Solids | mg/L | Monthly average | 10 or lower | < 10 |
| Total Coliform | MPN/100 mL | Weekly average | 2.2 | < 2.2 |
| Turbidity | NTU | 1 Hour average | 2 | < 2 |
| Settleable Solids | mg/L | Monthly average | 0.1 | 0.1 |
| Chlorine Residual | mg/L | 4-day average | 0.01 | 0.01 |
| Oil and Grease | mg/L | Monthly average | 10 | 2.1 |
| Aluminum | µg/L | Monthly average | 71 | 150 |
| Electrical Conductivity | µmhos/cm | Monthly average | 1000 | 825 |
| Ammonia[a] | mg/L | Monthly average | 2.1 | 1.5 |
| Ammonia[b] | mg/L | Monthly average | 2.8 | 1.5 |
| Arsenic | µg/L | Monthly average | 10 | 8 |
| Copper | µg/L | Monthly average | 7.9 | 7 |
| Cyanide | µg/L | Monthly average | 3.7 | 7 |
| Iron | µg/L | Monthly average | 300 | 47 |
| Manganese | µg/L | Monthly average | 50 | 15 |
| MBAS | µg/L | Monthly average | 500 | 560 |
| Nitrate (as N) | mg/L | Monthly average | 10 | 5 |
| Nitrite (as N) | mg/L | Monthly average | 1 | 1 |
| Bis (2-ethylhexyl) phthalate | µg/L | Monthly average | 22 | 3.48 |
| Bromodichloromethane | µg/L | Monthly average | 5 | ND[c] |
| Dibromochloromethane | µg/L | Monthly average | 1.4 | ND[c] |
| 2,4,6-Trichlorophenol | µg/L | Monthly average | 34 | 3.28 |

[a] June – September
[b] October – May
[c] Non-detect

## a. Achievable Limits

The Phase III Expansion project, particularly Schedule D improvements, have been designed to reliably reduce constituent concentrations to typical tertiary treatment levels for BOD, total suspended solids, total coliform, turbidity, settleable solids, chlorine residual, oil and grease, ammonia, and nitrogen. These tertiary treatment goals are consistent with limitations in the current NPDES permit and subsequent revisions can be readily achieved.

b. Limits That May Be Achievable

Reductions in contaminants such as arsenic and electrical conductivity may occur as the result of source water quality improvements. Well head treatment planned for the City water supply network may also contribute to improvements in background water quality. These represent important considerations because the current treatment process proposed at the WQCF is not designed to accomplish removal of these constituents. As such, reliable compliance with NPDES limits can only be ensured based on of the timing, extent, and effectiveness of alternate source water development.

c. Limits That Are Unlikely to Be Achieved

For metals and other organics, some reductions may occur as a by-product of the planned filtration-UV disinfection process in Schedule D based on historical performance at similar plants. Some cyanide reduction may be observed upon switching from chlorination to UV disinfection. Despite these potential reductions, compliance with the future NPDES limits for aluminum, cyanide, and MBAS will prove problematic. In general, because of the uncertainty in predicting removal efficiencies for metals and organics through the filtration process, consistent permit compliance with a number of the proposed effluent limits cannot be assured. Only membrane processes have demonstrated effective removal of dissolved contaminants for the levels identified in the current/future permit for surface water discharges.

## 2.6 Strategy for Future Compliance

Continued surface water discharge to the San Joaquin River represents a key component of the long-term wastewater management strategy for the City. For those constituent limits that may prove problematic such as aluminum, cyanide, and MBAS, continued sampling is recommended to better determine background levels and potential sources. Where appropriate, dissolved metal concentrations should be determined to then demonstrate no reasonable potential to exceed the BP objective. Additional data may demonstrate that constituent levels can be met through operational changes at the WQCF as may be the case for cyanide and MBAS. Where allowed by the RWQCB for aluminum and iron, site specific limits based on water effect ratio studies should be developed that better reflect BP objectives and the assimilative capacity of the receiving water. Site specific objectives are likely to be achieved for aluminum and iron based on historical effluent quality. The overall goal in terms of regulatory compliance should then be to implement pollution prevention programs (PPPs) that ensure continued discharge of filtered, disinfected effluent without the addition of membrane processes.

# 3 Projected Flow and Loadings

Anticipated future flow rates and loadings to the Manteca WQCF are presented below. Future average daily WQCF flows are predicted using General Plan land use information [4] and wastewater generation factors established for specific land uses. Future maximum daily and peak hourly WQCF flows are projected using peaking factors based on WQCF flow data in 2003. Projected flows are presented for Year 2023 (General Plan 20-year horizon) and buildout ultimate conditions.

## 3.1 Projected Flow

Wastewater generation factors (WGF) for various land uses are summarized in Table 3-1. Residential WGFs were calculated using the dwelling densities adopted in October 2003 [4]. Future average daily flows for each land use area are projected by multiplying the appropriate WGF with the land use area. As presented in Table 3-2, flows predicted for various land uses are then totaled to estimate the future contribution from the City of Manteca. Because of a contractual agreement between the Cities of Manteca and Lathrop, 14.7 percent of the WQCF capacity is made available for the treatment of Lathrop wastewater. Combined future Manteca and Lathrop flows are summarized in Table 3-3.

As presented in Table 3-3, future average wastewater flows for Manteca and Lathrop are anticipated at 19.5 mgd and 3.5 mgd in Year 2023, and 23 mgd and 4 mgd, at buildout, respectively. The total projected average wastewater flow to the WQCF is 23 mgd at Year 2023 and 27 mgd at buildout. Differences in flow from Year 2023 to buildout represent development within the Urban Reserve areas as detailed in the General Plan [4].

**TABLE 3-1**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**WASTEWATER GENERATION FACTORS**

| Land Use | Designation | Density[a] (dwelling units/acre) | Wastewater Generation Factor [b] (gpd/acre) |
|---|---|---|---|
| Very low density residential | VLDR | Less than 2 | 530[c] |
| Low density residential | LDR | 2.1-8 | 1,338[d] |
| Medium density residential | MDR | 8.1-15 | 2,183[e] |
| High density residential | HDR | 15.1-25 | 3,789[f] |
| Commercial mixed use | CMU | -- | 2,473 |
| General commercial | GC | -- | 1,120 |
| Heavy industrial | HI | -- | 2,010 |
| Light industrial | LI | -- | 2,010 |
| Public/Quasi-Public (P/QP) | PQP | -- | 425 |
| Park | P | -- | 400 |
| Agriculture | AG | -- | 0 |
| Open space | OS | -- | 0 |
| Neighborhood commercial | NC | -- | 1,120 |
| Business industrial park | BIP | -- | 1,330[g] |
| Urban reserve | UR | -- | 0 |
| Very low density residential – urban reserve | UR-VLDR | -- | 530 |
| Low density residential – urban reserve | UR-LDR | -- | 1,338 |
| Medium density residential – urban reserve | UR-MDR | -- | 2,183 |
| Commercial mixed use – urban reserve | UR-CMU | -- | 2,473 |
| General commercial – urban reserve | UR-GC | -- | 1,120 |
| Light industrial – urban reserve | UR-LI | -- | 2,010 |
| Public/Quasi-Public – urban reserve | UR-PQP | -- | 425 |
| Park – urban reserve | UR-P | -- | 400 |
| Agriculture – urban reserve | AG-UR | -- | 0 |
| Business industrial park – urban reserve | UR-BIP | -- | 1,330 |

[a] Based on the adopted 2003 General Plan [4]
[b] Nonresidential generation factors based on the 1993 Wastewater Collection Master Plan [5]
[c] Generation rate based on 265 gpd/edu [6] and density of 2 dwelling units per acre
[d] Generation rate based on 265 gpd/edu [6] and density of 5.05 dwelling units per acre
[e] Generation rate based on 189 gpd/edu [6] and density of 11.55 dwelling units per acre
[f] Generation rate based on 189 gpd/edu [6] and density of 20.05 dwelling units per acre
[g] Generation rate assumed comparable to business park (office professional)

**TABLE 3-2**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**PROJECTED WASTEWATER FLOWS – MANTECA ONLY**
**YEAR 2023 AND BUILDOUT CONDITIONS**

| Land Use | Area, Acres | Wastewater Generation Factor[a] (gpd/acre) | Wastewater Flow, mgd |
|---|---|---|---|
| VLDR | 928 | 530 | 0.49 |
| LDR | 6,474 | 1,338 | 8.66 |
| MDR | 462 | 2,183 | 1.01 |
| HDR | 420 | 3,789 | 1.59 |
| CMU | 532 | 2,473 | 1.31 |
| GC | 893 | 1,120 | 1.00 |
| HI | 953 | 2,010 | 1.92 |
| LI | 1,048 | 2,010 | 2.11 |
| PQP | 1,124 | 425 | 0.48 |
| P | 536 | 400 | 0.21 |
| AG | 3,956 | 0 | 0 |
| OS | 438 | 0 | 0 |
| NC | 153 | 1,120 | 0.17 |
| BIP | 211 | 1,330 | 0.28 |
| UR | 1,758 | 0 | 0 |
| **Year 2023 (rounded)** | **19,890** | | **19.5** |
| UR-VLDR | 996 | 530 | 0.53 |
| UR-LDR | 1,432 | 1,325 | 1.92 |
| UR-MDR | 20 | 1,701 | 0.04 |
| UR-CMU | 43 | 2,473 | 0.11 |
| UR-GC | 43 | 1,120 | 0.05 |
| UR-LI | 115 | 2,010 | 0.23 |
| UR-PQP | 12 | 425 | 0.005 |
| UR-P | 65 | 400 | 0.03 |
| UR-AG | 1789 | 0 | 0 |
| UR-BIP | 425 | 1,330 | 0.57 |
| **Buildout (rounded)** | **24,830** | | **23.0** |

**TABLE 3-3**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**PROJECTED WASTEWATER FLOWS**

| | Year 2023 | Buildout |
|---|---|---|
| City of Manteca | 19.5 | 23.0 |
| City of Lathrop[a] | 3.5 | 4.0 |
| **Total** | 23.0 | 27.0 |

[a] Based on contractual agreement for 14.7% of total plant capacity.

Hydraulic peaking factors were estimated using 2003 WQCF flow data. Average daily total plant and Lathrop flows are shown in Figure 3-1. For reference, the ADWF in 2003 was 5.81 mgd. The maximum daily flow (MDF) was observed on October 5, 2003 at 7.86 mgd. This represents a daily peaking factor (MDF/ADWF) of 1.36.

Hourly flow rates observed for 15 day periods during February 2003, May 2003, August 2003, and November 2003 were used to establish hourly peaking factors (see Appendix A). A summary of seasonal hourly flow rates is provided in Table 3-4. The maximum hourly flow (MHF) was observed on November 19, 2003 at 15 mgd. The hourly peaking factor (MHF/ADWF) ranged from 1.98 to 2.58. For planning purposes, an hourly peaking factor of 2.19 is recommended, representing the greatest "average" peak hour condition observed. For reference, the design peak hour flow factor for the Phase III Expansion Project is 1.94.

**TABLE 3-4**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**SUMMARY OF HOURLY FLOW RATES IN 2003**

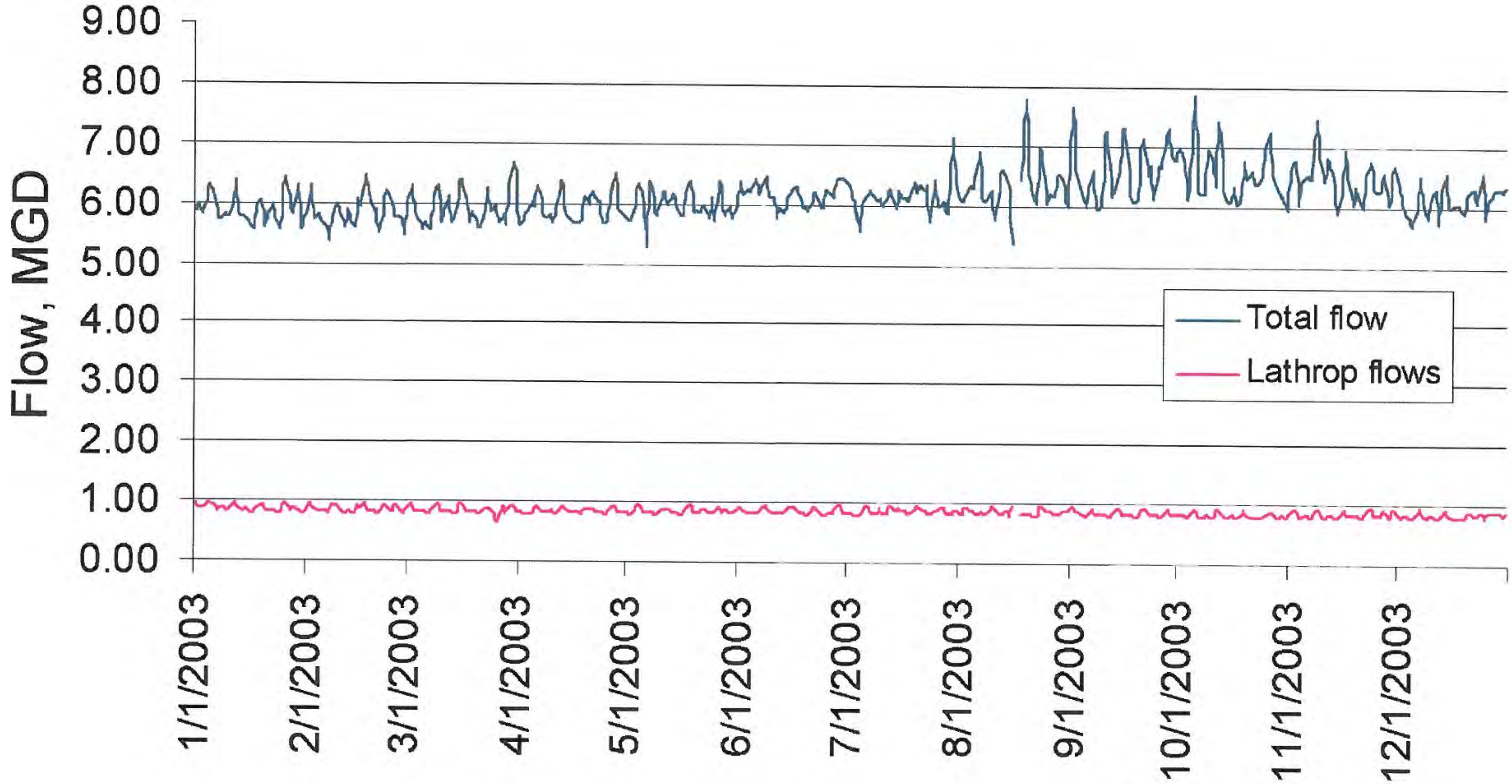
| | Maximum Hourly Flows (mgd) | | | Hourly Peaking Factor | | |
|---|---|---|---|---|---|---|
| | Average | Low | High | Average | Low | High |
| Winter [a] | 11.54 | 7.50 | 12.50 | 1.98 | 1.29 | 2.15 |
| Spring [b] | 12.46 | 12.00 | 13.50 | 2.14 | 2.06 | 2.32 |
| Summer [c] | 12.32 | 8.00 | 14.00 | 2.12 | 1.38 | 2.41 |
| Fall [d] | 12.75 | 11.00 | 15.00 | 2.19 | 1.89 | 2.58 |

[a] Based on WQCF flows between February 9, 2003 and February 22, 2003

[b] Based on WQCF flows between May 11, 2003 and May 24, 2003

[a] Based on WQCF flows between August 8, 2003 and August 24, 2003

[a] Based on WQCF flows between November 9, 2003 and November 22, 2003




| |
|---|
| FIGURE 3-1 |
| MANTECA WASTEWATER QUALITY CONTROL FACILITY<br>MASTER PLAN UPDATE |
| WQCF AVERAGE DAILY FLOW IN 2003 |
| NOLTE<br>BEYOND ENGINEERING |

Future maximum day and peak hour flows are estimated using the anticipated future average flows and the peaking factors discussed above. Future maximum daily flow and peak hourly flow are anticipated to be approximately 31.3 mgd and 50.1 mgd in Year 2023 and 36.7 mgd and 59.1 mgd at buildout, respectively. Influent flow rates observed in 2003 and anticipated future flow rates are summarized in Table 3-5.

**TABLE 3-5**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**SUMMARY OF WASTEWATER FLOWS (MGD)**

| Parameter | 2003 | 2023 | Buildout |
|---|---|---|---|
| Average dry weather flow | 5.81 | 23.0 | 27.0 |
| Maximum daily flow | 7.86 | 31.3 | 36.7 |
| Peak hourly flow | 15.0 | 50.1 | 59.2 |

[a] Flows from cities of Manteca and Lathrop.

## 3.2 Projected Loadings

Wastewater loadings are a key parameter in the development of design criteria for a wastewater treatment plant. For the Manteca WQCF, wastewater loadings used in design are summarized in Table 3-6. Future average and peak organics (BOD), TSS, and ammonia loadings (loadings) are based on plant influent data from January 2002 through January 2004 and the anticipated future ADWF to the plant. Wastewater loadings are then compared with design criteria for the Phase III Expansion Project.

**TABLE 3-6**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**SUMMARY OF WASTEWATER LOADINGS USED FOR DESIGN PURPOSES**

| Loading Parameter | Design Purpose |
|---|---|
| Average value | Quantify annual sludge production. Seasonal sludge production can also be estimated using average values.<br><br>Estimate annual operational and maintenance requirements and costs. |
| Maximum monthly or 95th percentile (whichever is higher) | Design of biological treatment processes such as aeration basins considering 30-day NPDES permit requirements. |
| Maximum weekly | Check secondary treatment design criteria (developed based on maximum monthly or 95$^{th}$ percentile values) and determine weekly effluent quality. Compare with 7-day NPDES permit requirements. Maximum weekly values would be used in design of secondary treatment facilities, if the ratio of maximum weekly to maximum monthly (or 95$^{th}$ percentile) values exceed two. |
| Maximum daily | Design of waste activated sludge (WAS) thickening facilities.<br><br>Design of secondary clarifiers and filtration facilities.<br><br>Check secondary treatment design criteria (developed based on maximum monthly or 95$^{th}$ percentile values) and determine daily effluent quality. Compare with maximum daily NPDES permit requirements. Maximum daily values would be used in design of secondary treatment facilities, if the ratio of maximum daily to maximum monthly (or 95$^{th}$ percentile) values exceed three for BOD and TSS loading, and two for ammonia loading, respectively. |

a. <u>Future Average WQCF Loadings</u>

Influent BOD, TSS, and ammonia concentrations from January 2002 through January 2004 are shown in Figures 3-2, 3-3, and 3-4, respectively. Average BOD, TSS, and ammonia concentrations during this period are 383 mg/L BOD, 340 mg/L TSS, and 34 mg/L ammonia, respectively. As a comparison, the design values for the Phase III Expansion Project are 310 mg/L BOD, 315 mg/L TSS, and 35 mg/L TKN. For reference, the Phase III Expansion Project design criteria reflect influent quality data for the period 1995-2000.

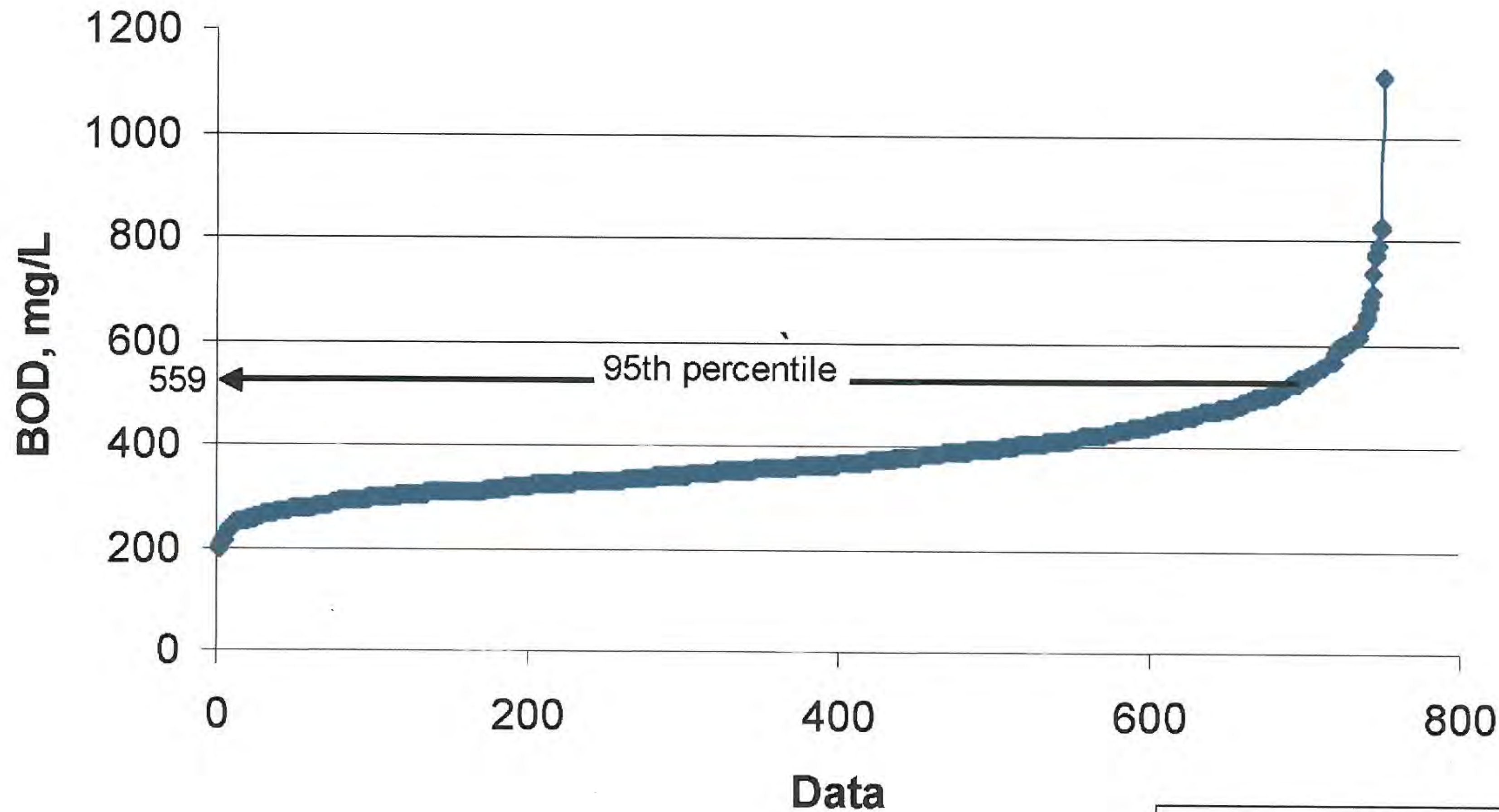



FIGURE 3-2

MANTECA WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE

DAILY AVERAGE BOD DATA
FROM JANUARY 2002 THROUGH JANUARY 2004

NOLTE
BEYOND ENGINEERING

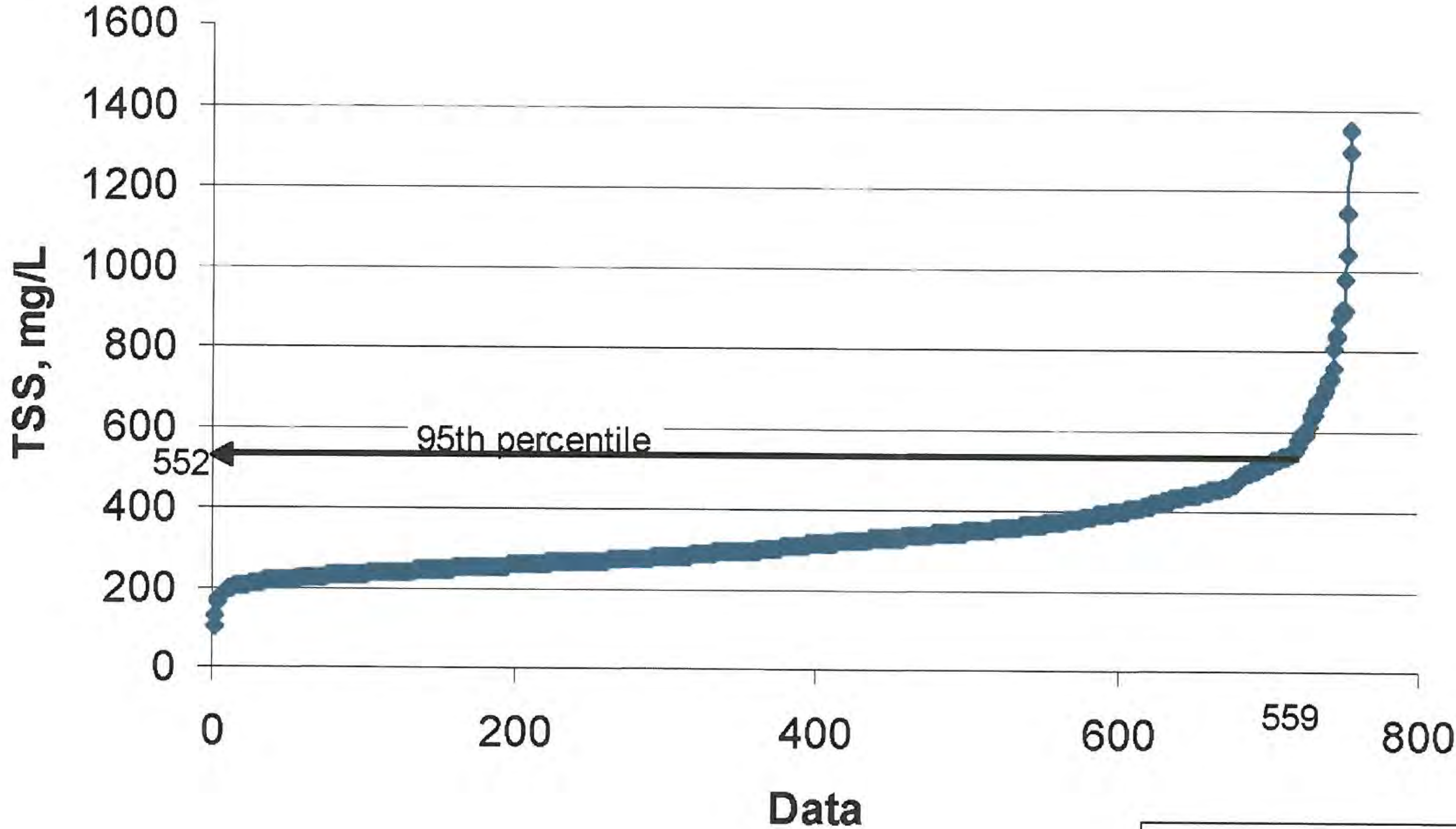



FIGURE 3-3

MANTECA WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE

DAILY AVERAGE TSS DATA
FROM JANUARY 2002 THROUGH JANUARY 2004

NOLTE
BEYOND ENGINEERING

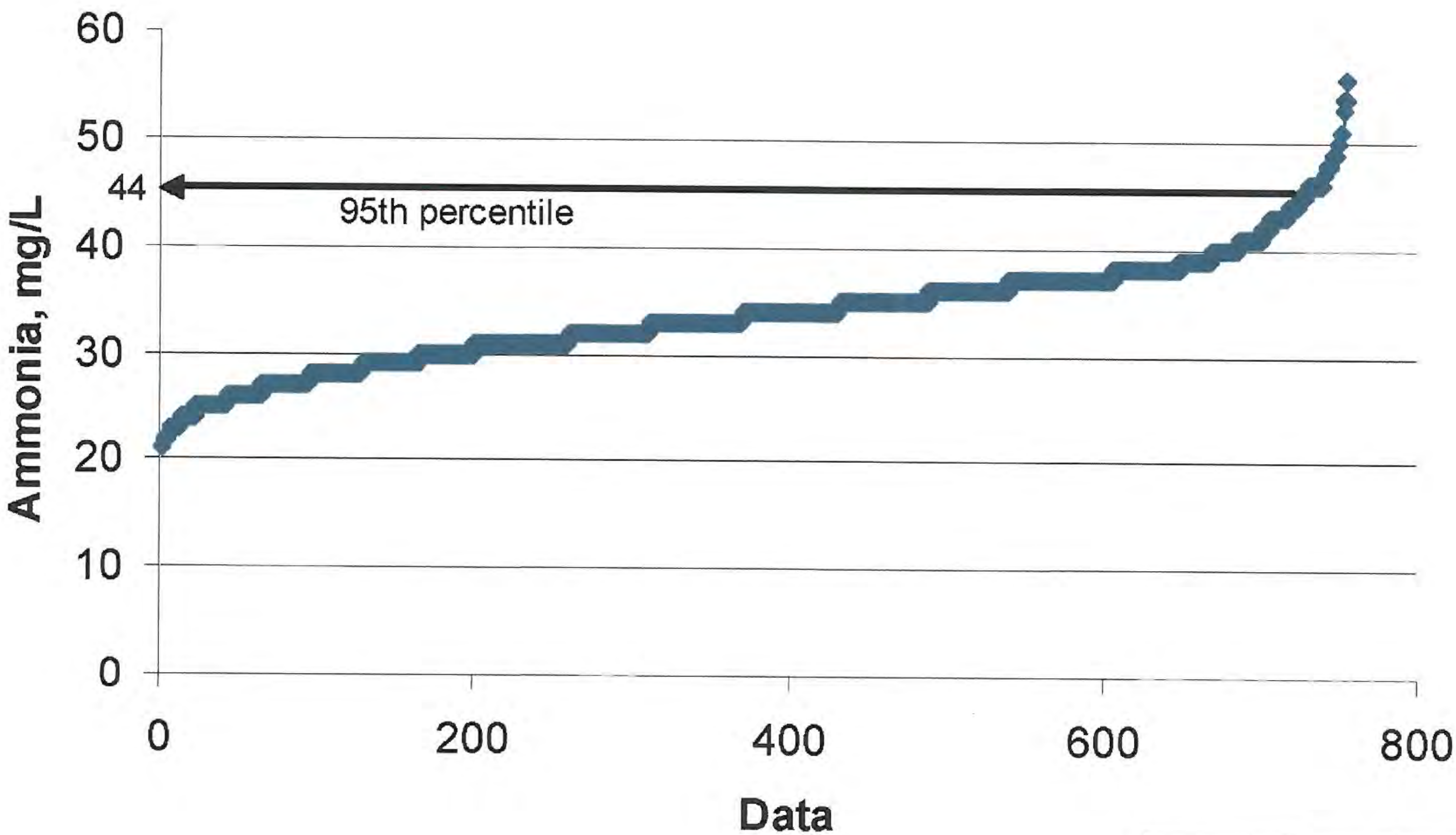



FIGURE 3-4

MANTECA WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE

DAILY AVERAGE AMMONIA DATA
FROM JANUARY 2002 THROUGH JANUARY 2004

NOLTE
BEYOND ENGINEERING

The average influent BOD and TSS values were observed to increase 24 percent and eight percent (compared to the Phase III Expansion Project design values), respectively in 2002 and 2003. The peak influent BOD and TSS values were also observed to increase 28 percent and 38 percent, respectively in 2002 and 2003. Primary effluent TSS and BOD values obtained for 2002 and 2003 were compared to 2001 and 1991-1992 primary effluent data to investigate the increase in influent values in 2002 and 2003. TSS and BOD values are summarized in Table 3-7.

**TABLE 3-7**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**COMPARISON OF HISTORICAL TSS AND BOD INFLUENT AND PRIMARY EFFLUENT VALUES**

| | Average TSS (mg/L) | | | Average BOD (mg/L) | | |
|---|---|---|---|---|---|---|
| **Period** | **Influent** | **Primary Effluent** | **Primary Removal Efficiency** | **Influent** | **Primary Effluent** | **Primary Removal Efficiency** |
| 1991 - 1992 | 221 | 68 | 69% | 262 | 178 | 32% |
| 2001 | 278 | 105 | 62% | 319 | 229 | 28% |
| January 2002 - December 2003 | 333 | 99 | 70% | 376 | 234 | 38% |

The following conclusions are offered from a review of the results presented in Table 3-7:

1. Average influent BOD and TSS increased 22 percent and 26 percent, respectively between 1991-1992 and 2001. Primary effluent BOD and TSS increased 29 percent and 54 percent, respectively. The increases in primary effluent BOD and influent BOD are consistent (22 percent versus 29 percent). The increase in primary effluent TSS is higher compared to the increase in influent TSS (54 percent versus 26 percent). This suggests a possible error in any one of the following measurements: (1) 1991 influent or primary effluent TSS or (2) 2001 influent or primary effluent TSS values.

2. Primary effluent values seem to remain unchanged from 2001 to 2002-2003. This suggests that measured influent TSS and BOD values observed in 2002 and 2003 may be higher (up to 20 percent) than the actual values. Therefore, 10 percent reduction in measured 2002 and 2003 BOD and TSS values is deemed to be appropriate for design purposes, as discussed below.

Based on a future flow of 27 mgd (ADWF), average loadings are presented in Table 3-8 considering the 10 percent reduction in 2002 and 2003 BOD and TSS values. Specifically, at buildout the anticipated average BOD, TSS, and ammonia loadings will be 78,400 lb/day, 70,000 lb/day, and 7,600 lb/day, respectively.

**TABLE 3-8**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**ANTICIPATED FUTURE LOADING AT PLANT BUILDOUT**

| | BOD | TSS | Ammonia |
|---|---|---|---|
| Influent Concentrations, mg/L [a] | | | |
| Average | 348 | 309 | 34 |
| Maximum monthly | 501 | 592 | 41 |
| 95th percentile | 506 | 511 | 44 |
| Maximum weekly | 614 | 780 | 46 |
| Maximum daily | 1,013 | 1,229 | 56 |
| Influent Loads, lb/day [b] | | | |
| Average | 78,400 | 70,000 | 7,600 |
| Maximum monthly | 113,000 | 133,500 | 9,200 |
| 95th percentile | 114,000 | 115,000 | 10,000 |
| Maximum weekly | 138,500 | 176,000 | 10,500 |
| Maximum daily | 228,500 | 277,000 | 12,700 |
| Peaking Factors | | | |
| Maximum monthly / average | 1.44 | 1.91[c] | 1.21 |
| $95^{th}$ percentile / average | 1.46[d] | 1.65 | 1.30[d] |
| Maximum weekly / average | 1.77 | 2.52 | 1.38 |
| Maximum daily / average | 2.91 | 3.98 | 1.66 |
| Maximum weekly / maximum monthly [e] | 1.23 | 1.32 | 1.14 |
| Maximum weekly / 95th percentile [e] | 1.21 | 1.53 | 1.06 |
| Maximum daily / maximum monthly [f] | 2.02 | 2.08 | 1.38 |
| Maximum daily / 95th percentile [f] | 2.00 | 2.41 | 1.27 |

[a] Based on WQCF data from January 2002 through January 2004 (after 10 percent reduction)
[b] Based on WQCF influent concentration data (10 percent reduction) from January 2002 through January 2004 and anticipated future ADWF of 27 mgd
[c] TSS design peaking factor (maximum monthly peaking factor >$95^{th}$ percentile peaking factor).
[d] BOD and ammonia design peaking factor
[e] Ratio is less than 2. Therefore, use of maximum monthly TSS value and $95^{th}$ percentile values for BOD and ammonia in design of secondary treatment facilities is appropriate
[f] Ratio is less than 3 for BOD and TSS and less than 2 for ammonia. Therefore, use of $95^{th}$ percentile values for BOD and ammonia and maximum monthly TSS value in design of secondary treatment facilities is appropriate

b. Peaking Factors

Concentration peaking factors were calculated using influent BOD, TSS, and ammonia concentrations from January 2002 through January 2004 (see Figures 3-2, 3-3, and 3-4). Loading peaking factors were also calculated using the influent concentrations and flow rates observed in 2002 and 2003. Loading peaking factors and concentration peaking factors were of similar magnitude. Concentration peaking factors were slightly higher. Therefore for master planning purposes, concentration peaking factors will be used for facility sizing.

A summary of peaking factors is presented in Table 3-9. As discussed in Table 3-6, either peaking factors based on 95$^{th}$ percentile values (value larger than 95 percent of all observed values) or maximum monthly (maximum 30-day running average) values, or a combination of both parameters, are used typically in design of secondary treatment processes. The 95$^{th}$ percentile peaking factors for BOD (1.46) and ammonia (1.30) are slightly larger than the monthly peaking factors and therefore will be established as the design peaking factors for BOD and ammonia. As seen in Table 3-9, the monthly peaking factor for TSS (1.91) is larger than the 95$^{th}$ percentile peaking factor (1.65) and therefore will be used as the design peaking factor for TSS. As a comparison, the design peaking factors for the Phase III Expansion Project are 1.4, 1.5, and 1.8 for BOD, TSS, and TKN respectively. As discussed above, peaking factors based on influent values are 1.46 and 1.91 for BOD and TSS, respectively. Monthly peaking factors based on primary effluent values are also presented in Table 3-9. The recommended peaking factors for the design of secondary treatment facilities are then given in Table 3-9. These peaking factors represent a conservative approach to facility sizing without excessive oversizing of individual unit pressures.

**TABLE 3-9**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**SUMMARY OF PEAKING FACTORS TO BE USED IN THE**
**DESIGN OF SECONDARY TREATMENT FACILITIES**

| | Phase III Expansion-Peaking Factors | Influent Peaking Factors[a] | Primary Effluent Peaking Factors[b] | Recommended Secondary Treatment Design Peaking Factor[c] |
|---|---|---|---|---|
| BOD | 1.4 | 1.46 | 1.29 | 1.4 |
| TSS | 1.5 | 1.91 | 1.46 | 1.7 |

[a] Based on 2002-2003 influent BOD and TSS data
[b] Based on 2002-2003 primary effluent BOD and TSS data
[c] Obtained by averaging the influent and primary effluent peaking factors. Peaking factors are then rounded.

c. Future Peak WQCF Loadings

Recommended peaking factors and anticipated future average loadings discussed previously are used to estimate the future peak loadings. As presented in Table 3-10, future maximum monthly BOD, TSS, and ammonia loadings are expected to be 114,000 lb/day, 133,500 lb/day, and

10,000 lb/day, respectively. Maximum daily BOD, TSS, and ammonia loadings are anticipated to be 228,000 lb/day, 279,000 lb/day, and 13,000 lb/day, respectively. As mentioned above, maximum daily values are considered in design of waste activated sludge (WAS) thickening and tertiary filtration facilities. Design values for primary and secondary treatment processes are summarized in Table 3-10.

**TABLE 3-10**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**SUMMARY OF ANTICIPATED FUTURE LOADINGS AT BUILDOUT**

| Constituent | Peaking Factor | Loading | |
|---|---|---|---|
| | | Average | Peak |
| **Primary Treatment Processes** | | | |
| BOD | 1.46 | 78,400 | 114,000 |
| TSS | 1.91 | 70,000 | 133,500 |
| Ammonia | 1.30 | 7,600 | 10,000 |
| **Secondary Treatment Processes** | | | |
| BOD | 1.40 | 78,400 | 110,000[a] |
| TSS | 1.70 | 70,000 | 119,000[a] |
| Ammonia | 1.30 | 7,600 | 10,000[a] |
| **WAS Thickening/Tertiary Filtration** | | | |
| BOD | 2.91 | 78,400 | 228,000[b] |
| TSS | 3.98 | 70,000 | 279,000[b] |
| Ammonia | 1.66 | 7,600 | 13,000[b] |

[a] Maximum monthly or 95$^{th}$ percentile
[b] Maximum daily

# 4 Effluent Disposal Strategies

The long-term recommended disposal strategy for the City WQCF is presented below. Treated effluent from the WQCF would be disposed of by three methods: 1) On-site land application; 2) Urban landscape irrigation; and 3) Discharge to the San Joaquin River. Off-site land application through an expanded agricultural irrigation program is not considered feasible because of the magnitude of the required land acquisition (greater than 5,000 ac) and the potential distance from the plant. The WQCF currently discharges to the San Joaquin River or on-site land application. A combination of the three disposal methods will be utilized to discharge the buildout flow of 27 mgd. Each disposal strategy, and the amount of treated effluent that can be discharged by that method, are described below and summarized in Table 4-1.

## 4.1 On-Site Land Application

The City currently discharges approximately 1030 ac-ft/yr (0.87 mgd) of undisinfected secondary effluent to 190-ac of City-owned land. An existing 9-ac, 15-Mgal storage pond located on City land immediately west of the WQCF is used in the on-site land application system. The City is planning to develop 30-ac of City-owned land for a state of the art softball complex, leaving 160-ac of City-owned land available for on-site land application.

The permeability of soils identified at the on-site land application area ranged from moderately rapid to rapid [7]. The hydraulic loading at the WQCF is generally between 65 and 70 inches per year. Assuming an application rate of 65 inches per year on the remaining 160-ac of City-owned land, approximately 0.73 mgd of treated effluent can be disposed of by on-site land application.

## 4.2 Urban Landscape Irrigation

The technical memorandum *Evaluation of Urban Water Recycling Opportunities from the Manteca Wastewater Quality Control Facility* [8] investigates the use of urban landscape irrigation to dispose of treated effluent from the WQCF. The technical memorandum identifies 741-ac of irrigable urban land. This information has subsequently been updated, and a total of 817-ac of irrigable urban land have been identified, including parks, schools, cemeteries, and golf courses [9]. Assuming an application rate of 54 in/yr, 3.28 mgd of recycled water would be discharged for urban landscape irrigation (see Appendix B).

It should be noted that unrestricted urban irrigation with recycled water requires the production of tertiary disinfected recycled water as defined by the DHS. This water quality requirement will be satisfied upon completion of the Phase III Expansion Project. To reduce the potential for public contact with recycled water, parks, schools, and cemeteries are assumed to be irrigated between 10 p.m. and 6 a.m. (This limited window for landscape irrigation is a typical DHS requirement.) The golf courses are assumed to receive water 24-hours per day.

## 4.3 River Discharge

The City is currently permitted to discharge 8.11 mgd to the San Joaquin River under NPDES Permit No. CA0081558 [2]. The permit allows the monthly average discharge to increase to 9.87 mgd in February 2009, provided that additional water quality requirements are satisfied by the WQCF. The increase in flow to 9.87 mgd will be discharged to the San Joaquin River through the existing 36-inch outfall. Due to concerns of thermal impacts to the San Joaquin River, the WQCF will likely be restricted from discharging during reverse flow events. Beyond 9.87 mgd ADWF, a second parallel outfall will be required to convey the balance of WQCF effluent either not land applied on-site or pumped off-site for urban irrigation. A discussion of the routing of the parallel 36-inch land outfall is presented in Appendix C.

**TABLE 4-1**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**RECOMMENDED DISPOSAL STRATEGY AT BUILDOUT**

| Disposal Option | Average Discharge Rate, mgd | Annual Volume, ac-ft/yr | Percentage of Total Discharge |
|---|---|---|---|
| On-Site Land Application | 0.73 | 870 | 3% |
| River Discharge | 22.99 | 25,760 | 85% |
| Urban Landscape Irrigation | 3.28 | 3,670 | 12% |
| Total | 27.00[a] | 30,300 | 100% |

[a] Based on anticipated wastewater flow to Manteca WQCF, see Chapter 3.

# 5 Odor Control

To develop an odor control strategy at the Manteca WQCF, it is important to understand historical odor control practices, potential odor sources, and odor control alternatives. Each topic is discussed below.

## 5.1 Historical Odor Control Practices

The historical odor control practice at the WQCF focused on reducing the dissolved sulfide concentration in the wastewater prior to the generation and release to the air of hydrogen sulfide ($H_2S$). Air was introduced to the influent trunk sewer via a blower at the influent pump station to move any odorous air away from the plant, considering the facility has no provisions to reduce or remove $H_2S$ once it has been released to the atmosphere. The Manteca WQCF began using ferric chloride for hydrogen sulfide control in the spring of 1996. The typical dosage was 1-3 mg/L. The ferric chloride was stored in a 6,500 gal portable tank near the septage receiving station and was added to the 36-in. North Manteca Trunk Sewer using a chemical metering pump. Dosage was usually manually adjusted with no automatic pacing. To avoid overdosing during low flow periods, chemical addition was discontinued from 12 a.m. to 6 a.m. typically each day. The combined practice of ventilation and the addition of ferric chloride proved relatively effective in reducing odors at the plant although chemical augmentation was suspected in the accelerated deterioration of influent pump impellers.

## 5.2 Potential Odor Services

Field investigations and wastewater sampling later confirmed the high potential for $H_2S$ odor generation at the Manteca plant. With the facility sited several miles from either the Lathrop or Manteca collection systems, detention times are long in trunk sewers and force mains, particularly during periods of low flow. Significant concentrations of hydrogen sulfide have been detected at the plant and future design conditions will likely not mitigate these existing problems. Because the new influent pump station constructed as part of the Phase III Expansion Project must accommodate buildout flows hydraulically, during current low flows, there may be drops in water surface elevations that contribute to off-gassing. With the new headworks/influent pump station constructed immediately adjacent to the easterly property line, there is little room for error in terms of odor dissipation. The large volume of air that must be circulated and vented from the pump station represents a significant source of odorous air that left untreated would likely lead to neighbor complaints. Considering these factors and the desire not to impact adjacent future development potential adversely, particularly with a new City softball complex, an odor control system is recommended for installation with the headworks/influent pump station and primary sedimentation basins.

## 5.3 Discussion of Odor Control Techniques

Ferric chloride additions have been successful in reducing the dissolved $H_2S$ concentrations in raw wastewater entering the plant. Expansion of the ferric chloride application program further upstream within the wastewater collection system (e.g., Union Road Pump Station) or at the Lathrop O Street pump station could help mitigate odor potential at the Manteca WQCF. However, considering the future re-routing of the Lathrop force main to a junction structure with the North Manteca Trunk Sewer, the range of flows and hydraulic conditions anticipated at the headworks/influent pump station, and the long detention times in the collection system, ferric chloride addition alone, will likely not be sufficient to control odors. Other odor control techniques therefore should be assessed as follows.

a. Description of Alternatives

There are three principle types of odor control techniques that have been used successfully for many years at wastewater treatment plants and numerous hybrid or experimental strategies that have a limited track record. This assessment will be limited to the three most common systems:

1. Packed Tower Scrubbing
2. Mist Scrubbing
3. Soil Filter

**Packed Tower Scrubbing:** Packed tower scrubbing relies on chemicals to react with the odor compounds found in the air collected from the process area. The packed tower is a vertical tank, usually constructed of fiberglass, that contains a bed of plastic packing sized from 1 in to 3 in. in nominal diameter. The chemical solution (sodium hydroxide) is pumped to the top of the media and flows over the large surface area of the media in a thin sheet. This creates a large surface area for the odor molecules to come in contact and react with the chemical solution. The air is drawn into the bottom section of the vertical tower with a fiberglass duct axial fan and exhaust is vented out the top of the tower.

The application rate of the chemical solution can be based on the pH and oxidation reduction potential of the liquid after it has passed over the plastic media and reacted with odor compounds. The chemical solution is collected in a sump and is pumped back over the top of the media. A portion of the chemical solution is routinely wasted and replaced with fresh solution to prevent a buildup of waste products.

**Mist Scrubbing:** The mist scrubbing system is similar to the packed tower in that it relies on a chemical solution and a large surface area on which the odor molecule can react with the chemical solution. In this technique the large surface area is achieved by creating a fine mist of the chemicals. The mist droplets have a high surface area to volume ratio. The nozzle that creates the mist has a very small opening. To avoid plugging the small nozzle opening with impurities from the foul air stream, the chemical solution comes in contact with the foul air only once. There is no recycling of the chemical solution. The spent solution is discharged to the wastewater stream.

The feed rate of the chemical solution is based upon the air flow rate and the concentration of the odor compounds in the foul air. Matching these two parameters requires a more complicated chemical feed and control system than the packed tower alternative. The removal efficiency of the mist scrubbing system is reduced by high and fluctuating levels of hydrogen sulfide.

**Soil Filter:** The soil filter, or biofilter, relies on bacteria in either wet compost, soil, or an engineered medium to remove the majority of the odor compound without the need to use large quantities of chemicals. The foul air is blown through the bottom of the soil filter through a distribution system of perforated pipe and as the air comes in contact with the bacteria, the hydrogen sulfide and other odor compounds are consumed. Water sprayed on top of the compost, soil, or engineered medium maintains the correct moisture for high odor removal rates. The byproduct produced by the bacteria is acidic, so the pH of the media decreases with use. A small quantity of neutralizing chemicals can be applied to keep the soil filter pH in the optimum range for removing odor compounds. The typical sizing criteria for biofilters is 0.5 $ft^2$ soil filter/cfm of air flow. Based on the required air flow of 13,000 cfm, a minimum 6,500 $ft^2$ biofilter would be necessary for odor control at the influent pump station. Soil filters are well suited for applications that require low air flow rates (1,500 cfm or less). Operational problems such as uneven air distribution and short circuiting have been reported at rates above 1,500 cfm, however, many large scale installations (greater than 10,000 cfm) have successful track records.

b. Comparison of Odor Control Techniques

With minimal mechanical equipment, the biofilter option represents the least capital investment although the first costs for the packed tower scrubber are not significantly greater. The wide divergence in costs between the options occurs when considering operation and maintenance requirements. Both the packed tower and mist scrubber systems rely upon the use of chemicals that create significantly higher annual costs than the biofilter. Considering this difference in annual costs, the biofilter alternative is the lowest cost option on a life-cycle basis. In view of this comparison, biofilters are recommended for odor control at the WQCF.

# 6 Solids Handling Practices

A discussion of solids handling practices of the WQCF is presented in this chapter. Projected loadings and disposal strategies are described below.

## 6.1 Description of Solids Handling Facilities

Solids handling facilities include sludge thickening, sludge digestion, and sludge dewatering. Each unit process is discussed below along with projected loadings at buildout.

### a. Sludge Thickening

Dissolved air flotation (DAF) will be used for the thickening of the waste activated sludge (WAS) produced in the secondary treatment facilities. The primary purpose of DAF is to reduce the liquid volume of WAS. Waste activated sludge and thickened waste activated sludge (TWAS) data observed at the Manteca WQCF between January 2003 and March 2004 were used to project the WAS and TWAS rates at buildout conditions. Dissolved air flotation units are designed for the observed TWAS flow daily peaking factor of 3.29. Average WAS and TWAS production at buildout is approximately 19,800 lb/day and 17,800 lb/day, respectively. Projected average and peak WAS and TWAS rates are summarized in Table 6-1. Peaking factors observed for the WAS rates were lower than the peaking factors observed for the TWAS rates. Observed TWAS peaking factors are used for planning purposes.

**TABLE 6-1**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**PROJECTED SOLIDS LOADING AT BUILDOUT**

| | | Primary Sludge (PS) | Waste Activated Sludge (WAS) | Thickened Waste Activated Sludge (TWAS) | Combined PS & TWAS (CS) |
|---|---|---|---|---|---|
| Peaking Factors | | | | | |
| Peak day | ratio | 4.71 | 3.29 | 3.29 | 3.91 |
| 15 day peak | ratio | 1.78 | 1.96 | 1.96 | 1.56 |
| Production | | | | | |
| Average | lb/day | 27,100 | 19,800 | 17,800 | 44,900 |
| Daily peak | lb/day | 127,400 | 65,000 | 58,500 | 185,900 |
| 15 day peak | lb/day | 48,300 | 38,700 | 34,800 | 83,100 |
| Flow | | | | | |
| Average | gpd | 93,300 | 525,300 | 48,100 | 141,400 |
| Daily Peak | gpd | 439,100 | 1,730,400 | 158,300 | 597,400 |
| 15-Day Peak | gpd | 166,200 | 1,029,800 | 94,200 | 260,400 |

b. Sludge Digestion

Sludge generated in the primary sedimentation basins and TWAS will be stabilized in the anaerobic digesters. The purpose of the anaerobic digestion process is to reduce the mass of solids, to stabilize the organic material, to reduce the pathogen content of the solids, and to produce a combustible gas that can be used for sludge heating and electricity generation. Primary sludge (PS) and TWAS data observed at Manteca WQCF between January 2003 and March 2004 were used to project the combined sludge (CS) rates to the anaerobic digesters at buildout conditions. Average PS and CS production for the expansion is anticipated to be approximately 27,100 lb/day and 44,800 lb/day, respectively.

Due to the relatively long detention times within the digesters, a 15 day peaking factor is recommended for sizing of the digestion facilities. Therefore, the observed 15 day peaking factor of 1.56 for the CS flow will determine the volume of any new digestion. Projected average and peak PS and CS loadings are also presented in Table 6-1.

c. Sludge Dewatering

High solids centrifugal dewatering systems will be used to dewater digested sludge. These systems have the following major advantages as compared to other dewatering alternatives: (1)

they produce the highest solids content sludge and (2) they have the least operational and maintenance requirements. Polymer will be injected into the sludge feed line to enhance the dewatering process.

Dewatered cake from the centrifuges will be discharged at a solids content of approximately 25 percent onto a solids handling conveyor. The conveyor will discharge cake to a biosolids composting and storage area. The discharge of the conveyor will be located at a height that can accommodate truck loading if the City elects to haul biosolids off site at a future time.

The centrifuges will be operated Monday through Friday approximately 10 hr/day for average conditions. The centrifuges will be operated Monday through Saturday approximately 16 hr/day during peak sludge production. To store digester effluent generated during the weekends and non-working weekday hours, construction of an equalization tank as part of the dewatering process is recommended.

Centrifuge centrate (the liquid discharge) will flow by gravity to a centrate equalization transfer structure located adjacent to the dewatering process building. From the transfer structure, the centrate will be discharged to a centrate equalization tank.

The purpose of the centrate equalization tank is to reduce shock loading of centrate to the biological treatment processes. Based on historical laboratory analyses of the decant from the existing sludge drying beds, it is projected that the centrate may have a TKN concentration ranging from 600-1,000 mg/L. At this concentration, if no equalization were provided, the centrate would increase the incoming nitrogen loading significantly. This shock loading would make consistent operation of the secondary biological treatment system difficult. With equalization, the centrate will be released back to the headworks at a constant rate minimizing potential shock loading.

## 6.2 Biosolids Disposal

Biosolids generated currently at the existing and Phase III expansion facilities will continue to be reused beneficially on the City-owned land surrounding the plant in an agronomic application. Due to concerns regarding the potential generation of odors from a biosolids stockpile, the solids handling facilities for the Phase III expansion are designed to facilitate truck loading for subsequent off-site disposal. Biosolids generated from the ultimate (27 mgd) facility will be disposed at an off-site landfill.

# 7 Recommended Facility Improvements

Wastewater treatment facilities designed to meet the anticipated water quality objectives described in Chapter 2 are discussed in this chapter. A site plan at buildout is included as Figure 7-1. Design criteria are presented typically for the incremental expansion required to increase the facility capacity from 10.0 to 27.0 mgd. Overall hydraulic loading and solids loading criteria are summarized in Tables 7-1 and 7-2. Individual unit processes and design criteria are discussed as follows.

**TABLE 7-1**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR PLANT EXPANSION**
**HYDRAULIC LOADING**

| Parameter | Unit | Buildout | Incremental Expansion |
|---|---|---|---|
| **Flow** | | | |
| Average dry weather flow, MGD | mgd | 27.0 | 17.5 |
| Maximum daily flow | mgd | 36.7 | 23.8 |
| Peak hourly flow | mgd | 59.2 | 38.4 |

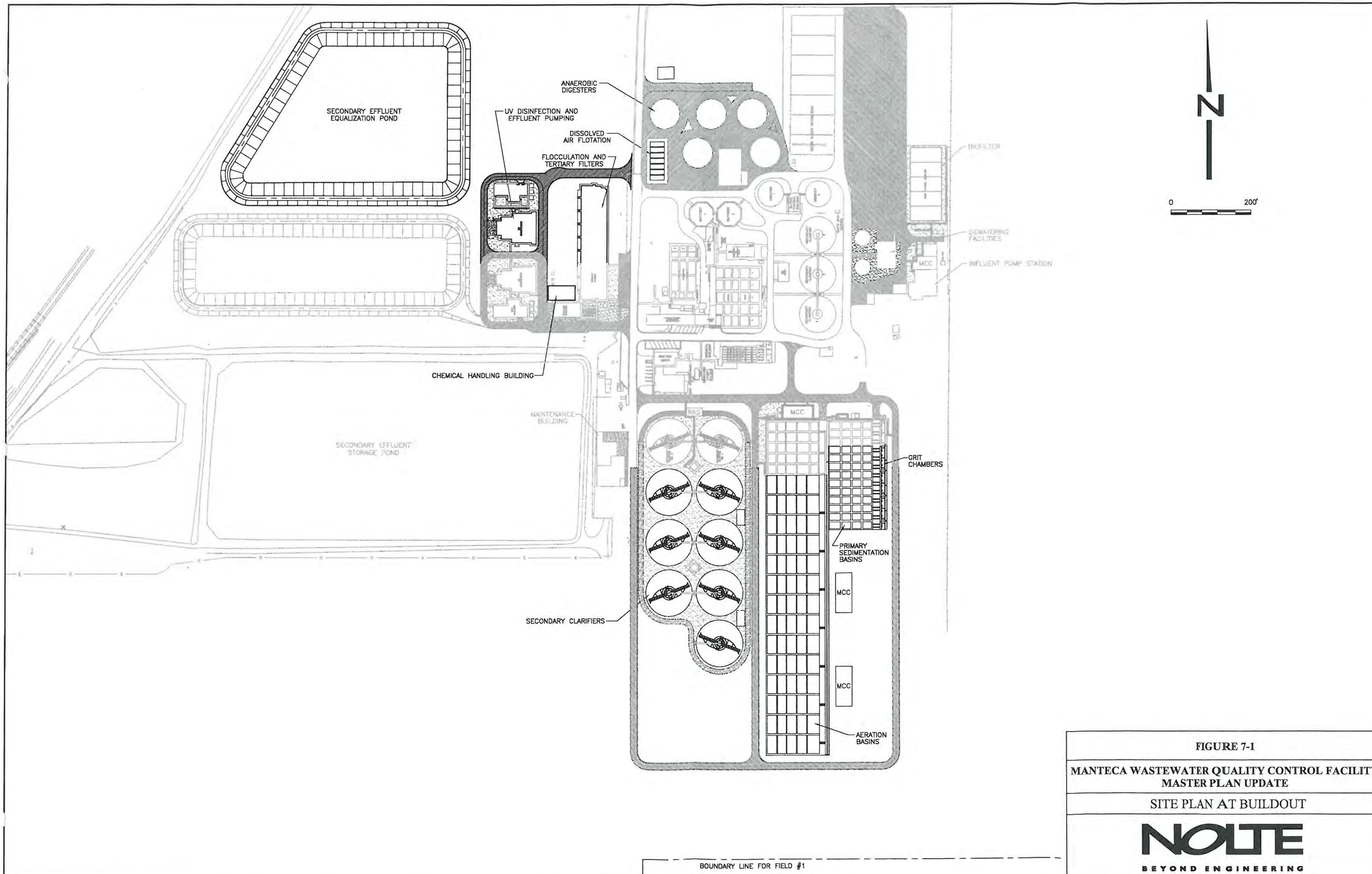



**FIGURE 7-1**

**MANTECA WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN UPDATE**

SITE PLAN AT BUILDOUT

NOLTE
BEYOND ENGINEERING

**TABLE 7-2**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**SUMMARY OF INFLUENT LOADINGS**

| Parameter | Unit | BOD | TSS | Ammonia |
|---|---|---|---|---|
| **Influent Concentrations** | | | | |
| Average | mg/L | 348 | 309 | 34 |
| Maximum monthly | mg/L | 501 | 592 | 41 |
| Maximum weekly | mg/L | 614 | 780 | 46 |
| Maximum daily | mg/L | 1,013 | 1,229 | 56 |
| 95th percentile | mg/L | 506 | 511 | 44 |
| **Influent Loads (Total at Buildout)** | | | | |
| Average | lb/day | 78,400 | 70,000 | 7,600 |
| Maximum monthly | lb/day | 113,000 | 133,300 | 9,200 |
| Maximum weekly | lb/day | 138,500 | 176,000 | 10,500 |
| Maximum daily | lb/day | 228,500 | 277,000 | 12,700 |
| 95th percentile | lb/day | 114,000 | 115,100 | 10,000 |
| **Influent Loads (Incremental Expansion)**[a] | | | | |
| Average | lb/day | 50,800 | 45,500 | 5,000 |
| Maximum monthly | lb/day | 73,300 | 86,400 | 6,000 |
| Maximum weekly | lb/day | 89,800 | 114,100 | 6,800 |
| Maximum daily | lb/day | 148,200 | 179,500 | 8,200 |
| 95th percentile | lb/day | 73,900 | 74,600 | 6,500 |

[a] Incremental expansion represents the difference between buildout and Phase III values.

## 7.1 Influent Pump Station/Mechanical Screening

As part of several long-term infrastructure improvements, the City is constructing two new trunk sewers to serve the north and south areas of the community, respectively. The new trunk sewers will be sized to convey the projected buildout flow of the community. Modification of the existing influent pump station to receive future flows was not economically or technically feasible due to the lower inverts of the new trunk sewers and the physical space limitations associated with the existing pump station. Therefore, a new influent pump station will be constructed as part of the Phase III improvements.

The influent pump station is sited on the easterly perimeter of the WQCF property to allow for easy tie-in with the new trunk sewers while minimizing disruption to existing facilities and structures. The station includes two mechanical screens, a wet well, a dry well equipped with four centrifugal pumps, and a flow meter. Two fine mechanical screens (3/8 inch openings) are designed to remove coarse solids and non-biodegradable materials such as plastics. Removal of these materials will protect the influent pumps and subsequent downstream equipment as well as improve the quality of the biosolids produced at the plant (i.e., plastics in biosolids are highly undesirable when applied as a soil conditioner). Space is provided for a third mechanical screen for the buildout plant conditions.

Four screw centrifugal pumps will deliver screened raw wastewater to the NSF and SSF, respectively. Discharge from two 100-hp pumps will be conveyed to the NSF. Discharge from one 200-hp pump will be conveyed to the SSF. Through the operation of valves, a standby 200-hp pump can discharge to either the NSF or SSF. Space is provided in the influent pump station to install four additional pumps to handle projected future flows. Screw centrifugal pumps were selected because of their ability to pass solids and their high efficiency. Odor control will be accomplished by routing foul air to a central biofilter.

## 7.2 Grit Removal/Primary Sedimentation

Seven aerated grit tanks and ten primary sedimentation basins are proposed for the incremental expansion of the SSF headworks and primary treatment unit processes.

### a. Aerated Grit Tanks

The purpose of the grit tanks is to remove relatively high density inorganic particles (e.g., sand) from the wastewater stream, thus reducing wear and tear on downstream equipment. The grit tanks were sized to provide sufficient detention time to allow the separation of the grit from the wastestream. Typical detention times in aerated grit tanks range from 2 to 5 minutes at peak flow [9].

The proposed grit tanks for the expansion facilities are designed with a 4 minute detention time at the peak flow (3.5 minutes with one unit out of service) as summarized in Table 7-3.

**TABLE 7-3**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**AERATED GRIT TANKS**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| Number of new tanks | no | 7 | | |
| Number of Phase III SSF tanks | no | 2 | | |
| Length | ft | 24 | | |
| Width | ft | 8 | | |
| Depth (nominal) | ft | 10.5 | | |
| Detention time | min | | 8.7 | 4.0 |
| Detention time (one unit out of service) | min | | 7.7 | 3.5 |
| Airflow, total | cfm | | 840 | 840 |

The 3:1 length to width ratio of the grit tanks (24 ft long x 8 ft x 10.5 ft deep) along with the placement of coarse bubble aerators along one side of the tank creates a spiral flow pattern resulting in the separation of the heavy grit particles from the wastewater. Recommended aeration rates in aerated grit tanks range from 3 to 8 cfm/ft of length [9]. A total airflow of 840 cfm to the grit tanks is suggested to supply an aeration rate of 5 cfm/ft.

Once separated from the wastewater, grit will settle into the hoppers in the bottom of the tanks. Grit collected in the hoppers will be discharged to a grit classifier by gravity for further concentration and washing of light organic material trapped with the grit. Concentrated and washed grit discharged from the classifier will be conveyed into a dumpster and periodically hauled to the landfill for disposal. Wash water from the classifier will be discharged into the plant drain system and returned to the influent pump station (IPS).

b. Primary Sedimentation Basins

The purpose of the primary sedimentation basins (PSBs) is to remove settleable solids and floatable scum, usually oil and grease. This is accomplished by providing a quiescent environment where particles with densities greater than water can settle and those with densities less than water will float to the surface.

The design of PSBs can be based on detention time or overflow rates. Recommended detention times for PSBs are 1.5 to 2.5 hours [11]. Typical overflow rates range from 800 to 1,200 gpd/ft$^2$ under average flow to 2,000 to 3,000 gpd/ft$^2$ under peak flow conditions [9]. Detention times and overflow rates for the PSBs are presented in Table 7-4. It can be seen in Table 7-4 that the detention time and overflow rates in the proposed PSBs will range from 1.2 to 2.8 hours and 776 to 1,842 gpd/ft$^2$ depending upon the flow regime and number of basins in service. While under certain conditions detention times in the PSBs are slightly below recommended values, overflow rates, a more critical indicator of process performance, always fall within the suggested criteria. To mitigate potential odors, the PSBs will be covered, with foul air routed to a local biofilter.

**TABLE 7-4**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**PRIMARY SEDIMENTATION BASINS**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| BOD removal in PSBs | percent | 30 | | |
| TSS removal in PSBs | percent | 60 | | |
| Overflow rate, all PSBs in service | gpd/ft$^2$ | | 776 | 1,700 |
| Overflow rate, one PSB out of service | gpd/ft$^2$ | | 841 | 1,842 |
| Number of Phase III SSF PSBs | no | 3 | | |
| Number of new PSBs | no | 10 | | |
| Length to width ratio | ratio | 5.4 | | |
| Length of each PSB | ft | 110 | | |
| Width of each PSB | ft | 20 | | |
| Total area of PSBs | ft$^2$ | 22,544 | | |
| Water depth | ft | 12 | | |
| Detention time | hr | | 2.8 | 1.3 |
| Detention time (one PSB out of service) | hr | | 2.6 | 1.2 |

## 7.3 Aeration Basins with Anoxic Zone

Seven new aeration basins, each rated for an average flow rate of 2.50 mgd are proposed. Each basin is approximately 126 ft long, 92 ft wide, with a side water depth of 17 ft. Each basin is divided into five zones, separated by baffle walls. The baffle walls create a serpentine flow pattern reducing the potential for hydraulic short-circuiting. The basins will be operated in a plug flow fashion with an initial anoxic zone, followed by four aerated zones. The ratio of the anoxic zone to the total volume is approximately 23 percent.

Carbonaceous BOD removal and nitrification will be accomplished in the aerated zones. Nitrified wastewater will be recirculated back to the first anoxic zone for nitrate removal, which is known as the modified Ludzack-Ettinger (MLE) process. Without aeration, the first zone in each basin will become anoxic. Under anoxic conditions, nitrate produced in the aerated zones will be reduced to nitrogen gas to achieve the required degree of total nitrogen removal. Other benefits of an initial anoxic zone include the recovery of alkalinity consumed during nitrification and suppressing the growth of filamentous bacteria. The design of the aeration basin was based on cold weather conditions to consider worst case conditions. As seen in Table 7-5 for cold weather conditions, internal recirculation ratio varies between approximately 290 percent for average load conditions to 410 percent during peak load conditions. Typical internal recirculation ratios for MLE processes vary between 200 and 400 percent. Internal recirculation pumps will be sized for a flow rate of 11.5 MGD for each aeration basin to accommodate peak flow rates.

Other important criteria for the design of BOD and nitrogen removal facilities include hydraulic retention time (HRT), mean cell residence time (MCRT), activated sludge production, mixed liquor suspended solids concentration (MLSS), and specific denitrification rate. As seen in Table 7-5, the HRT values range from 6.1 to 14.2 hours (depending on the flow regime and number of basins in service), well within the recommended range of 5 to 15 hours for MLE processes. The minimum suggested MCRT values to achieve complete nitrification for the temperatures observed at WQCF are 6 days and 12 days during warm weather and cold weather, respectively. As presented in Table 7-5, the MCRT values range from 12 days to 15 days (depending on loading conditions) satisfying the minimum suggested values for cold weather period. The sludge yield coefficient during cold weather was estimated to be approximately 0.28 and was used to calculate the activated sludge production in the aeration basins. Average activated sludge production (during cold weather) in the aeration basins varies between approximately 15,000 lb/day and 24,000 lb/day depending on loading conditions. Typical MLSS concentration in MLE processes ranges between 3,000 and 4,000 mg/L. The MLSS concentration is assumed to vary between 3,000 mg/L and 4,300 mg/L for average and peak loading conditions, respectively at Manteca WQCF. The specific denitrification rate was calculated to be 0.28 lb of nitrate conversion for lb of active MLSS concentration in the anoxic zone (in a day). Active MLSS concentration in the anoxic zone was calculated to vary between approximately 820 mg/L and 1,140 mg/L for the other MLE process design criteria presented above. Total oxygen requirement was calculated to vary between approximately 63,000 lb/day and 84,500 lb/day (depending on the loading conditions) as presented in Table 7-5.

**TABLE 7-5**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**AERATION BASIN**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| **Aerated Zone** | | | | |
| BOD design peaking factor | ratio | | | 1.4 |
| TSS design peaking factor | ratio | | | 1.7 |
| Ammonia design peaking factor | ratio | | | 1.3 |
| Organic nitrogen conversion factor | ratio | 1.15 | | |
| BOD concentration to aeration basin | mg/L | | 244 | 341 |
| TSS concentration to aeration basin | mg/L | | 124 | 210 |
| Ammonia concentration to aeration basin | mg/L | | 39 | 50 |
| BOD filtrate return concentration | mg/L | 600 | | |
| Ammonia filtrate return concentration | mg/L | 100 | | |
| Assumed sludge filtrate return flow | gpm | 200 | | |
| BOD to aeration basins | lb/day | | 36,500 | 50,800 |
| Ammonia to aeration basins | lb/day | | 6,000 | 7,700 |
| BOD removal in secondary | percent | 97 | | |
| Ammonia removal in secondary | percent | 100 | | |
| TSS removal in secondary | percent | 80 | | |
| Solids retention time | day | | 15.0 | 12.0 |
| MLSS concentration | mg/L | | 3,000 | 4,300 |
| Return activated sludge ratio | percent | | 50 | 60 |
| TSS volatile fraction | percent | 80 | | |
| TSS volatile biodegradable fraction | percent | 67 | | |
| Yield coefficient-heterotrophic | gVSS/gBOD | 0.28 | | |
| Decay coefficient-heterotrophic | 1/day | 0.10 | | |
| Yield coefficient-autotrophic | gVSS/gAmmonia | 0.12 | | |
| Decay coefficient-autotrophic | 1/day | 0.06 | | |
| Fraction cell debris | percent | 0.15 | | |
| Total activated sludge production | lb/day | | 14,600 | 23,800 |
| Nonbiodegradable VSS and inert TSS portion | lb/day | | 6,696 | 11,384 |
| Aerated volume | MG | 8.0 | | |
| Detention time (aerated zone) | hr | | 10.9 | 5.0 |
| Number of aerated compartments | no | 4 | | |

**TABLE 7-5 (CONTINUED)**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**AERATION BASIN**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| **Aerated Zone (cont.)** | | | | |
| Oxygen requirement | | | | |
| For BOD | lb-oxygen/lb-load | 1.0 | | |
| For Ammonia | lb-oxygen/lb-load | 4.6 | | |
| Total oxygen requirement | lb/day | | 63,000 | 84,500 |
| Volumetric BOD loading | lb BOD/1,000 $ft^3$-day | | 25 | 37 |
| Recycle ratio | percent | | 50 | 60 |
| Food to microorganism ratio | 1/day | | 0.20 | 0.22 |
| **Anoxic zone** | | | | |
| Ratio of anoxic zone volume | percent | 23 | | |
| Volume | MG | 2.4 | | |
| Detention time | hr | | 3.3 | 1.5 |
| Nitrate to anoxic zone | mg/L | | 35.3 | 45.7 |
| Effluent nitrate | mg/L | 8.0 | | |
| Internal recycle ratio for denitrification | percent | | 291 | 411 |
| Influent nitrate load to the anoxic zone | lb/day | | 4,000 | 5,500 |
| Anoxic zone active biomass, mg/L | mg/L | | 820 | 1,150 |
| Food to active microorganism ratio | 1/day | | 2.2 | 2.2 |
| Specific denitrification rate | lb $NO_3$/lb MLVSS -day | | 0.28 | 0.28 |
| Amount of nitrate that can be reduced | lb/day | | 4,550 | 6,400 |
| **Combined (aerated and anoxic zones)** | | | | |
| Total volume | MG | 10.3 | | |
| Number of new basins | no | 7 | | |
| Number of Phase III SSF basins | no | 5 | | |
| Length to width ratio | ratio | 7.6 | | |
| Depth, each | ft | 17.0 | | |
| Width, each | ft | 92.0 | | |
| Length, each | ft | 126.0 | | |
| Volume, each | MG | 1.48 | | |
| Detention time | hr | | 14.2 | 6.5 |
| Detention time, with one basin out of service | hr | | 12.2 | 5.4 |

## 7.4 Secondary Clarification

Mixed liquor from the aeration basins will be conveyed to secondary clarifiers for sludge separation before filtration and disinfection. Some of the settled sludge will be recycled back to the aeration basins to maintain the desired MLSS concentration levels in the aeration basins. Suggested recycle ratios for MLE processes are from 50 to 100 percent of the influent flow. The design criteria presented in Table 7-5 are for recycle ratios between 50 and 60 percent. The amount of sludge wasted from the secondary clarifiers to the solids handling facilities will be approximately equal to the amount of sludge produced in the aeration basins to maintain the desired MCRT values.

Seven new 110-ft secondary clarifiers with water depths of 16 ft are proposed for the incremental plant expansion. The secondary clarifiers are sized based on recommended overflow rates and solids loading rates for nitrifying activated sludge systems. The overflow rates and solids loading rates in these clarifiers are presented in Table 7-6. Typical overflow rates for secondary clarifiers range from 400 to 700 gpd/ft$^2$ and from 1,000 to 1,600 gpd/ft$^2$ at average and peak flow, respectively [11]. As presented in Table 7-6 the overflow rates vary between approximately 260 and 650 gpd/ft$^2$ depending on the number of clarifiers in service and the flow regime. Typical solids loading rates for secondary clarifiers are from 1.0 to 1.5 lb/ft$^2\cdot$hr and less than 1.8 lb/ft$^2\cdot$hr at average and peak flow, respectively [11]. As presented in Table 7-6 the solids loading rates vary between 0.4 and 1.5 lb/ft$^2\cdot$hr depending on the number of clarifiers in service and loading conditions. Typical depths range from 12 to 20 feet. Deeper depths are recommended for improved performance and a larger margin of safety when upsets occur in the activated-sludge system.

**TABLE 7-6**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**SECONDARY CLARIFICATION**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| Overflow rate | gpd/ft$^2$ | | 260 | 575 |
| Overflow rate, one clarifier out of service | gpd/ft$^2$ | | 295 | 650 |
| Number of new clarifiers | no | 7 | | |
| Number of Phase III SSF clarifiers | no | 2 | | |
| Diameter of each clarifier | ft | 110 | | |
| Total area of new clarifiers | ft$^2$ | 66,652 | | |
| Water depth, ft | ft | 16 | | |
| Solids loading rate | lb/hr-ft$^2$ | | 0.4 | 1.4 |
| Solids loading rate, with one clarifier out of service | lb/ft$^2$-hr | | 0.5 | 1.5 |

## 7.5 Return Activated Sludge/Waste Activated Sludge Handling

A return activated sludge (RAS) pumping station consisting of seven 15 to 20-hp RAS pumps (six duty and one standby) will provide RAS pumping (see Table 7-7). Each RAS pump is sized to provide a recycle flow up to 4.5 mgd at 15 ft total dynamic head (TDH). The total design RAS flow of approximately 27 mgd is equal to 70 percent of the peak design flow of 38.4 mgd. If necessary, the standby RAS pump is available to increase the total RAS flow rate to 31.5 mgd achieving a RAS ratio of approximately 80 percent at peak flow.

WAS will be conveyed to seven DAF units for thickening prior to anaerobic digestion. WAS will be directed to the DAF units from the discharge side of the RAS pumps. Under average conditions, the RAS pumps will provide sufficient pressure to convey WAS to the DAF units without supplemental pumping. However, under periods of increased wasting, booster pumps will be required to convey the WAS to the DAF units. Six one-hp inline booster pumps will be used for this purpose. A seventh booster pump will be in place for redundancy.

**TABLE 7-7**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**RETURN ACTIVATED SLUDGE PUMPING**

| Parameter | Unit | Average Conditions | Peak Conditions |
|---|---|---|---|
| Maximum design return ratio | percent | 100 | 70 |
| Maximum design return flow rate | mgd | 17.5 | 26.9 |
| Pump TDH | ft | 15.0 | 15.0 |
| Number of duty pumps | no | 6.0 | 6.0 |
| Pump efficiency | percent | 70 | 70 |
| Pump horsepower, each | hp | 11.0 | 16.8 |

## 7.6 Effluent Filtration

Nine effluent filters are proposed for the incremental plant expansion (beyond Phase III) to remove the remaining total suspended solids in the secondary effluent . Filters will be cloth disk type, designed to comply with the turbidity requirements of the Title 22 wastewater reuse criteria. The daily average filtered effluent turbidity will be less than 2 nephelometric turbidity unit (NTU). Filter effluent turbidity objectives in accordance with the Title 22 requirements are summarized in Table 7-8. Filtration will also increase the downstream disinfection efficiency by removing the suspended solid particles that are disinfection obstacles.

**TABLE 7-8**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**TERTIARY FILTRATION AND UV DISINFECTION OBJECTIVES**
**AS DEFINED IN TITLE 22 CRITERIA**

| Parameter | Value |
|---|---|
| Turbidity | 24 hour average ≤ 2 NTU<br>95 percent of values ≤ 5 NTU<br>At any time < 10 NTU |
| Total coliform bacteria | Running 7 day median ≤ 2.2 MPN/100 mL<br>No more than one event in<br>30 days ≥ 23 MPN/100 mL<br>At any time < 240 MPN/100 mL |

The total filtration surface area for the incremental expansion is approximately 5,800 $ft^2$. As presented in Table 7-9 filtration rate varies between 2.1 and 4.9 gpm/$ft^2$ depending on the flow regime and number of filters in service. Typical maximum filtration rate for cloth type disk filters is between 5 and 6.5 gpm/$ft^2$. The backwash reject rate is anticipated to be between three and five percent.

**TABLE 7-9**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**FILTRATION**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| Incremental filtration area required | $ft^2$ | 5,554 | | |
| Total number of Phase III units | no | 6 | | |
| Total number of new units | no | 9 | | |
| Number of filter disks per unit | no | 12 | | |
| Diameter of individual filter disk | ft | 6 | | |
| Filtration Rate | gpm/$ft^2$ | | 2.1 | 4.6 |
| Filtration rate during one filter out of service | gpm/$ft^2$ | | 2.2 | 4.9 |
| Backwash water reject ratio | percent | | 2 - 5 | 4 - 7 |

## 7.7 Rapid Mixing and Flocculation

Chemical addition prior to filtration may be necessary especially during high solids loading conditions to achieve the Title 22 turbidity requirements. Therefore, chemical addition facilities are included to add coagulants and polymers prior to the filters. Two rapid mixing tanks and six flocculation tanks will be constructed to provide efficient mixing of chemicals and to form larger size flocs that are easily filterable. As seen in Table 7-10, the rapid mix tanks will be square in shape, 11 ft by 11 ft. The flocculation tanks will also be square in shape, 31.5 ft by 31.5 ft.

**TABLE 7-10**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**RAPID MIX AND FLOCCULATION**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| **Rapid Mix Tank** | | | | |
| Detention Time | sec | | 132 | 60 |
| Number of new tanks | no | 2 | | |
| Volume per tank | gal | 13,400 | | |
| Sidewater depth | ft | 15 | | |
| Required surface area per tank | $ft^2$ | 119 | | |
| Length | ft | 11 | | |
| Width | ft | 11 | | |
| **Flocculation Tanks** | | | | |
| Detention Time | min | | 55 | 25 |
| Number of new tanks | no | 6 | | |
| Volume per tank | gal | 111,100 | | |
| Sidewater depth | ft | 15 | | |
| Required surface area per tank | $ft^2$ | 990 | | |
| Length | ft | 31.5 | | |
| Width | ft | 31.5 | | |

## 7.8 UV Disinfection/Effluent Pumping

Filtered effluent must be disinfected to comply with the Title 22 wastewater reuse criteria for unrestricted reuse. Disinfected effluent coliform objectives in accordance with the Title 22 requirements are summarized in Table 7-10. For reasons of worker safety, environmental protection, and operational flexibility, an ultraviolet light disinfection system will be used. The installation will be a low-pressure high-intensity UV system. The pressure and intensity refer to the type of UV lamps used. Design criteria for the UV disinfection system are summarized in Table 7-11. Sizing of the UV system is based on guidelines published by NWRI as implemented by the California DHS. Four new UV channels (beyond Phase III) are proposed for the incremental plant expansion. Similar to the initial UV disinfection/effluent pumping facility, the expanded system will also include effluent pumps to convey tertiary-treated wastewater through a new parallel 36-inch outfall.

**TABLE 7-11**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**UV DISINFECTION**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| UV transmittance | % | 55 | | |
| Minimum dose | uWs/cm$^2$ | 100,000 | | |
| Number of Phase III channels | no | 2 | | |
| Number of new channels | no | 4 | | |
| Banks, per channel | no | 5 | | |
| Lamps, total | no | 4,480 | | |
| Output, per lamp | W | 150 | | |
| Power consumption | kW | | 593 | 1,300 |

## 7.9 Chemical Handling, Storage, and Metering

A chemical handling, storage, and metering facility will be required to support the tertiary filtration process as coagulant and polymer may be needed occasionally for filter aids. The equipment in the facility will include four bulk tanks and one day-tank for coagulant, metering pumps for the coagulant, space for four polymer tanks or totes, and a polymer dilution unit. Sizing of the metering pumps will accommodate peak dosing conditions. Suggested chemical doses are presented in Table 7-12. All tanks and metering pumps will be installed indoors, in a CMU building to prevent damage due to sunlight and temperature extremes.

**TABLE 7-12**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**SUGGESTED CHEMICAL DOSAGES FOR FILTER AIDS**

| | | Dosage | |
|---|---|---|---|
| Chemical | Unit | Typical | Maximum |
| Coagulant | mg/L | 5-15 | 25 |
| Polymer | mg/L | 0.05-0.10 | 0.20 |

## 7.10 Dissolved Air Flotation (WAS Thickening)

Seven new DAF units are required for incremental plant expansion for WAS thickening. Sludge handling facilities are designed based on increased sludge production rates to provide flexibility for changes in the operational and process parameters. The observed heterotrophic yield coefficient of 0.28 was increased by approximately 50 percent for the design of the sludge handling facilities. Assuming a heterotrophic yield coefficient of 0.42, average and (daily) peak design WAS rates for the ultimate expansion facilities are anticipated to be approximately 19,800 lb/day and 65,000 lb/day, respectively. The design criteria for the DAF facilities are presented in Table 7-13. The DAF units are designed based on suggested maximum solid loading rate of 20 lb/ft$^2$-day [11]. As presented in Table 7-13 solid loading rates vary between 6.2 and 23.9 lb/ft$^2$-day depending on loading conditions and number of DAF units in service. At the higher loading rates, additional chemical usage may be required to achieve desired solids concentrations.

**TABLE 7-13**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**DISSOLVED AIR FLOTATION (WAS THICKENING)**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| Daily peaking factor | ratio | | | 3.29 |
| WAS loading | lb/day | | 19,700 | 65,000 |
| Loading rate | lb/ft$^2$-day | | 6.2 | 20.5 |
| Loading rate with one unit out of service | lb/ft$^2$-day | | 7.3 | 23.9 |
| Total new DAF area required | ft$^2$ | 3,162 | | |
| Number of new units | no | 7 | | |
| Length to width ratio | ratio | 3 | | |
| Area of one unit | ft$^2$ | 452 | | |
| Width of each unit | ft | 12 | | |
| Length of each unit | ft | 36 | | |

## 7.11 Anaerobic Digestion and Cogeneration

As discussed previously, wastewater sludge generated in the primary sedimentation basins and activated sludge process will be anaerobically digested. New anaerobic digester facilities proposed as part of the incremental expansion include five digesters. New digesters will be 74 ft in diameter and 45 ft in height. The total minimum effective volume is 4.18 MG. Sizing of the new digesters is based upon solids loading and hydraulic detention time criteria. Recommended values for solids loading are between 0.1 and 0.3 lb VSS/ft$^3$·d [9]. The average and peak (based on 15 day peaking factor) CS loadings to the anaerobic digesters are approximately 45,000 lb/d and 83,000 lb/day, respectively. Based on historical data, approximately 83 percent of the solids are volatile. Therefore, the volatile solids loadings will be approximately 37,000 lb/day and 58,000 lb/day for average and peak conditions, respectively. As seen in Table 7-14, volatile suspended solids (VSS) loading will range from 0.08 to 0.13 lb VSS/$ft^3$·d depending on the loading conditions. Typical values for hydraulic detention time are between 15 and 20 days [9]. Detention time will vary between 13 and 24 days depending on the loading conditions and the number of the digesters in service.

Anaerobic sludge digestion requires heating. The microorganisms responsible for the anaerobic digestion process operate most effectively at temperatures between 90 and 95°F. Sludge heating in the new digesters will occur in a similar fashion to the existing digesters. Sludge will be drawn off the digester mixing pipeline with a booster pump, conveyed through a spiral heat exchanger, and discharged back into the mixing pipeline.

**TABLE 7-14**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**ANAEROBIC DIGESTION (SLUDGE DIGESTION)**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| 15 day peaking Factor | | | | 1.56 |
| CS loading | lb/day | | 44,900 | 83,100 |
| Flow, gal/day | gpd | | 141,300 | 220,000 |
| Volatile solids percentage | percent | 82 | | |
| Dry volatile solids loading | lb/day | | 36,900 | 57,600 |
| Number of new digesters | no | 5 | | |
| Diameter of each digester | ft | 74 | | |
| Effective minimum depth | ft | 26 | | |
| Volume, total | MG | 4.18 | | |
| Volume, each | MG | 0.84 | | |
| Detention time (all digesters in service) | days | | 23.7 | 15.2 |
| Detention time (with one digester out of service) | days | | 20.3 | 13.0 |
| Volatile solids loading | lb VSS/day-$ft^3$ | | 0.08 | 0.13 |
| Volatile solids destruction, percent | percent | | 62 | 56 |

## 7.12 Dewatering Facilities

Anaerobically digested sludge will be dewatered in high solids centrifugal systems. Design criteria for the dewatering process are summarized in Table 7-15.

As a part of the Phase III improvements, a new centrifuge unit will be installed within a new dewatering building. The dewatering building will accommodate two 100-hp centrifuges at the completion of Phase III improvements with space for a new centrifuge unit for the incremental expansion. The centrifuges will be mounted on a platform within the building. To enhance the dewatering process, polymer will be injected into the sludge feed line.

Dewatered cake from the centrifuges will be discharged at a solids content of approximately 25 percent onto a solids handling conveyor. The conveyor will discharge cake outside the building onto a biosolids composting and storage area. As noted earlier, the centrifuges will be operated approximately 10 hr/day Monday through Friday for average conditions, and approximately 16 hr/day Monday through Saturday during peak sludge production. Considering these operational conditions, dewatered cake loading on the conveyor belt will vary between 8.6 and 8.9 tons/hour for the total plant flow (see Table 7-15). To store digester effluent generated during the weekends and non-working weekday hours, construction of an equalization tank is required. The size of the equalization tank is based on the need to store digested sludge from 5:00 p.m. on Friday to 8:00 a.m. on Monday or 63 hours. At the projected expansion CS flow rate of approximately 141,300 gal/day, the equalization tank requires a volume of 371,000 gal. An additional 15 percent storage volume is recommended as a factor of safety bringing the size of the equalization tank to 430,000 gal.

**TABLE 7-15**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**DESIGN CRITERIA FOR INCREMENTAL PLANT EXPANSION**
**CENTRIFUGE (SLUDGE DEWATERING)**

| Parameter | Unit | Value | Average Conditions | Peak Conditions |
|---|---|---|---|---|
| Weekly peaking factor | | | | 1.82 |
| Total number of Phase III units | no | 2 | | |
| Number of new units | no | 1 | | |
| Conveyor belt width | ft | 2 | | |
| Hours of operation in a day | hr | | 10 | 16 |
| Days of operation in a week | days | | 5 | 6 |
| Centrifuge flow rate | gpm/unit | | 165 | 156 |
| Centrifuge solids loading | lb/unit-hour | | 1,527 | 1,589 |
| Dewatered solids content | Percent | 25 | | |
| Belt solids loading (total plant) | tons/hr | | 2.1 | 2.2 |
| Belt dewatered cake loading (total plant) | tons/hr | | 8.6 | 8.9 |

## 7.13 Phasing of Treatment Plant Improvements

To accommodate future increases in flow conveniently, two phases of construction are anticipated at the Manteca WQCF. Under Phase IV, the facility will be expanded to a treatment capacity of 17.5 mgd (ADWF), with Phase V representing buildout conditions (27 mgd ADWF). Site plans for Phase IV and Phase V are included as Figures 7-2 and 7-3, respectively. Two phases of construction are recommended also to minimize the impact on plant operations.

**Figure 7-2 Site Plan at Phase IV**

**Figure 7-3 Site Plan at Phase V**

# 8 Probable Construction Costs

This chapter summarizes the probable construction costs for the incremental expansion of the Manteca WQCF from approximately 10 mgd to 27 mgd. Unit costs are included in Appendix D. As presented in Table 8-1, treatment plant costs are projected at approximately $99 million with a parallel 36-inch outfall anticipated cost of $4.3 million (see Appendix E). Total incremental expansion costs are projected at approximately $103 million.

**TABLE 8-1**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**PROBABLE CONSTRUCTION COSTS TO EXPAND FACILITY**
**FROM 10 MGD TO 27 MGD**

| Unit Process | Probable Construction Costs |
|---|---|
| Influent Pump Station | 1,985,000 |
| Grit Removal/Primary Sedimentation | 7,100,000 |
| Aeration Basins | 16,072,000 |
| Secondary Clarification | 8,357,000 |
| Tertiary Filtration | 5,829,000 |
| UV Disinfection/Effluent Pumping | 9,870,000 |
| Equalization Pond | 2,303,000 |
| Chemical Building | 1,291,000 |
| Anaerobic Digestion / Sludge Heating | 9,985,000 |
| Thickening Facilities | 2,840,000 |
| Dewatering Facilities | 1,119,000 |
| Odor Control | 1,659,000 |
| Subtotal | 68,410,000 |
| Civil/Site Improvements | 3,421,000 |
| Yard Piping | 6,841,000 |
| Electrical | 20,523,000 |
| Subtotal, Treatment Facilities | 99,195,000 |
| Outfall, 36-inch | 4,300,000 |
| Grand Total | 103,495,000 |

Expansion of the Manteca WQCF to build-out capacity is anticipated in two phases, Phase IV (17.5 mgd capacity) and Phase V (22 mgd capacity). A breakdown of probable construction costs by phase is included as Table 8-2.

**TABLE 8-2**
**CITY OF MANTECA**
**WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN**
**PROBABLE CONSTRUCTION COSTS TO EXPAND FACILITY FROM 10 MGD TO 27 MGD**

**COST BREAKDOWN BY PHASE**

| | Probable Construction Costs | | |
|---|---|---|---|
| **Unit Process** | **Total** | **Phase IV** | **Phase V** |
| Influent Pump Station | 1,985,000 | 1,985,000 | -- |
| Grit Removal/Primary Sedimentation | 7,100,000 | 3,550,000 | 3,550,000 |
| Aeration Basins | 16,072,000 | 9,184,000 | 6,888,000 |
| Secondary Clarification | 8,357,000 | 4,775,000 | 3,582,000 |
| Tertiary Filtration | 5,829,000 | 2,915,000 | 2,914,000 |
| UV Disinfection/Effluent Pumping | 9,870,000 | 7,403,000 | 2,467,000 |
| Equalization Pond | 2,303,000 | 2,303,000 | -- |
| Chemical Building | 1,291,000 | 1,291,000 | -- |
| Anaerobic Digestion / Sludge Heating | 9,985,000 | 5,991,000 | 3,994,000 |
| Thickening Facilities | 2,840,000 | 1,623,000 | 1,217,000 |
| Dewatering Facilities | 1,119,000 | 1,119,000 | -- |
| Odor Control | 1,659,000 | 1,659,000 | -- |
| Subtotal | 68,410,000 | 43,798,000 | 24,612,000 |
| Civil/Site Improvements (5%) | 3,421,000 | 1,711,000 | 1,710,000 |
| Yard Piping (10%) | 6,841,000 | 4,789,000 | 2,052,000 |
| Electrical (30%) | 20,523,000 | 14,366,000 | 6,157,000 |
| Subtotal, Treatment Facilities | 99,195,000 | 64,664,000 | 34,531,000 |
| Outfall, 36-inch | 4,300,000 | 4,300,000 | -- |
| Grand Total | 103,495,000 | 68,964,000 | 34,531,000 |

## REFERENCES


[1] *City of Manteca, Public Facilities Implementation Plan*, prepared by Nolte and Associates, December 1993.

[2] California Regional Water Quality Control Board, Central Valley Region, *Waste Discharge Requirements for City of Manteca, City of Lathrop and Dutra Farms Wastewater Quality Control Facility, San Joaquin County*, March 2004.

[3] California Regional Water Quality Control Board, Central Valley Region, *The Water Quality Control Plan (Basin Plan) – The Sacramento River Basin and the San Joaquin River Basin*, September 2004.

[4] *City of Manteca General Plan 2023*, adopted October 6, 2003.

[5] *Sewer Master Plan for City of Manteca Public Facilities Implementation Plan*, prepared by Nolte and Associates, December 1993.

[6] *City of Manteca, Wastewater Collection System Master Plan Wastewater Generation Factor Technical Memorandum*, prepared by Nolte Associates, February 2004.

[7] *Technical Memorandum 3-4, Evaluation of Rapid Infiltration on City-Owned Land Surrounding the Manteca Wastewater Quality Control Facility*, prepared by Nolte Associates, March 2002.

[8] *Technical Memorandum No. 3-2, Evaluation of Urban Water Recycling Opportunities from the Manteca Wastewater Quality Control Facility*, prepared by Nolte Associates, February 2002.

[9] Metcalf and Eddy, Inc., Wastewater Engineering: Treatment, Disposal, and Reuse, Third Edition, McGraw-Hill, 1991.

[10] United States Environmental Protection Agency, Process Design Manual for Nitrogen Control, October 1975.

[11] Water Environment Federation, Design of Municipal Wastewater Treatment Plants, Manual of Practice No. 8, 1992.

# APPENDIX A
# HOURLY FLOW RATES

**Table A-1**
**Summary of WQCF Hourly Flows in 2003**

| | Maximum Hourly Flows | | | | Peaking Factor | | | |
|---|---|---|---|---|---|---|---|---|
| | Average | Minimum | Maximum | Standard Deviation | Average | Minimum | Maximum | Standard Deviation |
| Winter [a] | 11.54 | 7.50 | 12.50 | 1.25 | 1.98 | 1.29 | 2.15 | 0.21 |
| Spring [b] | 12.46 | 12.00 | 13.50 | 0.54 | 2.14 | 2.06 | 2.32 | 0.09 |
| Summer [c] | 12.32 | 8.00 | 14.00 | 1.45 | 2.12 | 1.38 | 2.41 | 0.25 |
| Fall [d] | 12.75 | 11.00 | 15.00 | 1.01 | 2.19 | 1.89 | 2.58 | 0.17 |

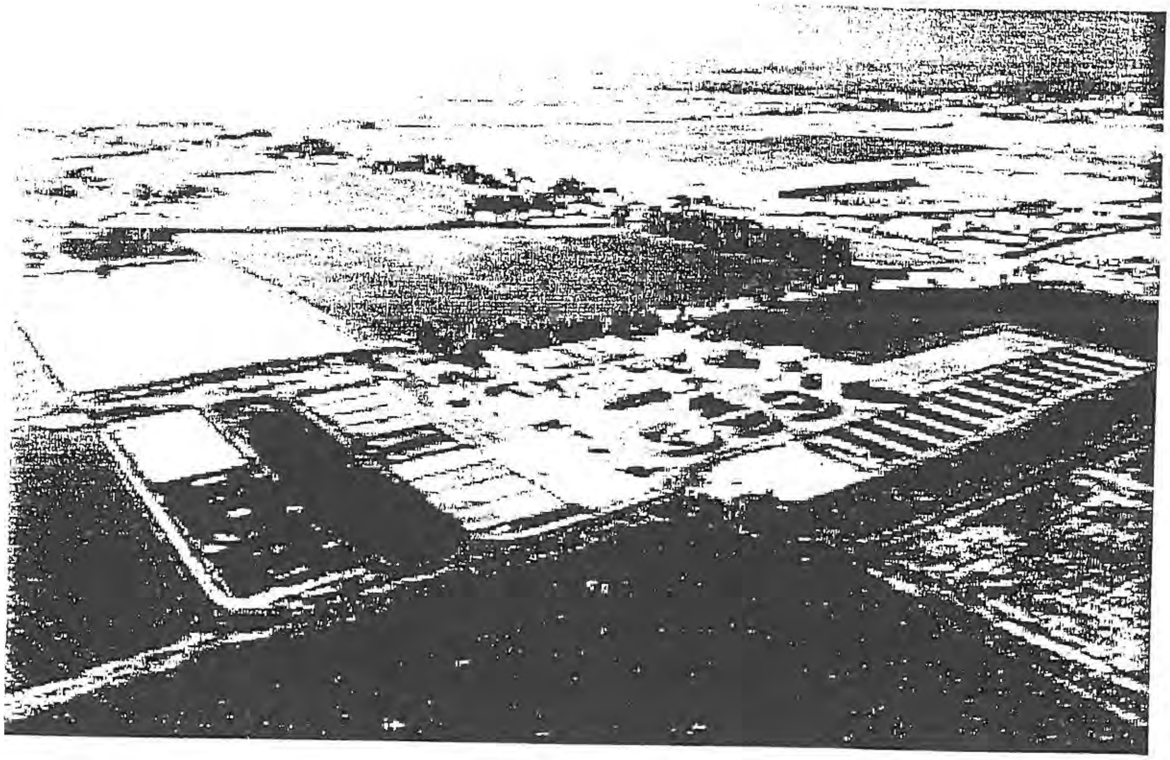
[a] Based on WQCF flows between February 9, 2003 and February 22, 2003
[b] Based on WQCF flows between May 11, 2003 and May 24, 2003
[a] Based on WQCF flows between August 8, 2003 and August 24, 2003
[a] Based on WQCF flows between November 9, 2003 and November 22, 2003

# APPENDIX B
# URBAN WATER RECYCLING OPPORTUNITIES

# CITY OF MANTECA
# WASTEWATER QUALITY CONTROL FACILITY
# MASTER PLAN UPDATE

## UPDATE OF URBAN WATER RECYCLING OPPORTUNITIES

## TECHNICAL MEMORANDUM



SEPTEMBER 2004

NOLTE

BEYOND ENGINEERING

# CITY OF MANTECA
# WASTEWATER QUALITY CONTROL FACILITY
# MASTER PLAN UPDATE

# UPDATE OF URBAN WATER RECYCLING OPPORTUNITIES

# TECHNICAL MEMORANDUM

SEPTEMBER 2004

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CITY OF MANTECA
WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE

# UPDATE OF URBAN WATER RECYCLING OPPORTUNITIES

# TECHNICAL MEMORANDUM

SEPTEMBER 2004

The purpose of this memorandum is to update urban water recycling opportunities within the City of Manteca (City). Potential urban reuse sites are identified and recycled water demands estimated. A conceptual distribution system is developed to serve these sites.

## Background

The use of recycled water for unrestricted urban landscape irrigation represents an alternative for treated wastewater disposal. Effluent from the Manteca Wastewater Quality Control Facility (WQCF) is currently either land applied to approximately 360 acres of City owned/leased parcels or discharged to the San Joaquin River. The volume of effluent from the WQCF will increase as the City population increases. Discharges to existing land application sites are currently constrained. Due to the implementation of the California Toxics Rule and Total Maximum Daily Limits (TMDLs) for the San Joaquin River, continued river discharge may result in the need for expensive advanced treatment systems (e.g. ultrafiltration and reverse osmosis). Urban and agricultural land application of recycled water could augment the City's existing land application practices thus reducing reliance upon surface water discharges. The use of recycled water to meet landscaping demands would have the additional benefit of reducing demand on the City's potable water system. A review of agricultural land application sites was conducted recently in a separate technical memorandum [1]. This memorandum will review urban landscape irrigation sites.

## Previous Reclamation Studies

Urban water recycling at the WQCF has been examined previously. The results of these previous studies are summarized below and provide a basis for this current examination of water recycling.

Manteca Effluent to Land Disposal Study, Technical Memorandum No. 3-2, Evaluation of Urban Water Recycling Opportunities from the Manteca Wastewater Quality Control Facility

Urban water recycling opportunities within the City were evaluated in 2002. The evaluation identified one hundred thirteen potential sites, ninety-four of which seemed economically feasible based on proximity to the WQCF and location near other sites. The total area of the ninety-four reuse sites was 935 acres, 711 acres of which were irrigable and could potentially receive 3,200 ac-ft/yr of recycled water. The projected cost of construction for delivery facilities was $10.1 million. With the inclusion of operation and maintenance costs, the cost for recycled water delivery was estimated at $423/ac-ft.

<u>1996 Phase III Pre-Design Water Reuse Investigation</u>

An urban water reuse investigation was conducted at the WQCF in January 1996 [3]. In that investigation potential reuse sites were identified following a windshield survey of the community and a review of future development plans for areas south of State Route 120 (SR-120). The field survey focused upon parks, schoolyards, golf courses, and greenbelts that could be converted from potable water to recycled water for nonpotable demands. Sixty-three reuse sites with a total annual recycled water demand of approximately 2,700 ac-ft/yr were identified. Treatment and storage facilities were sized and costs were estimated to meet a maximum daily recycled water demand of 5.7 mgd. A conceptual recycled water distribution system was also sized and costs were estimated to meet peak hour demands. The investigation concluded that the life cycle cost of supplying recycled water would be approximately $470/ac-ft.

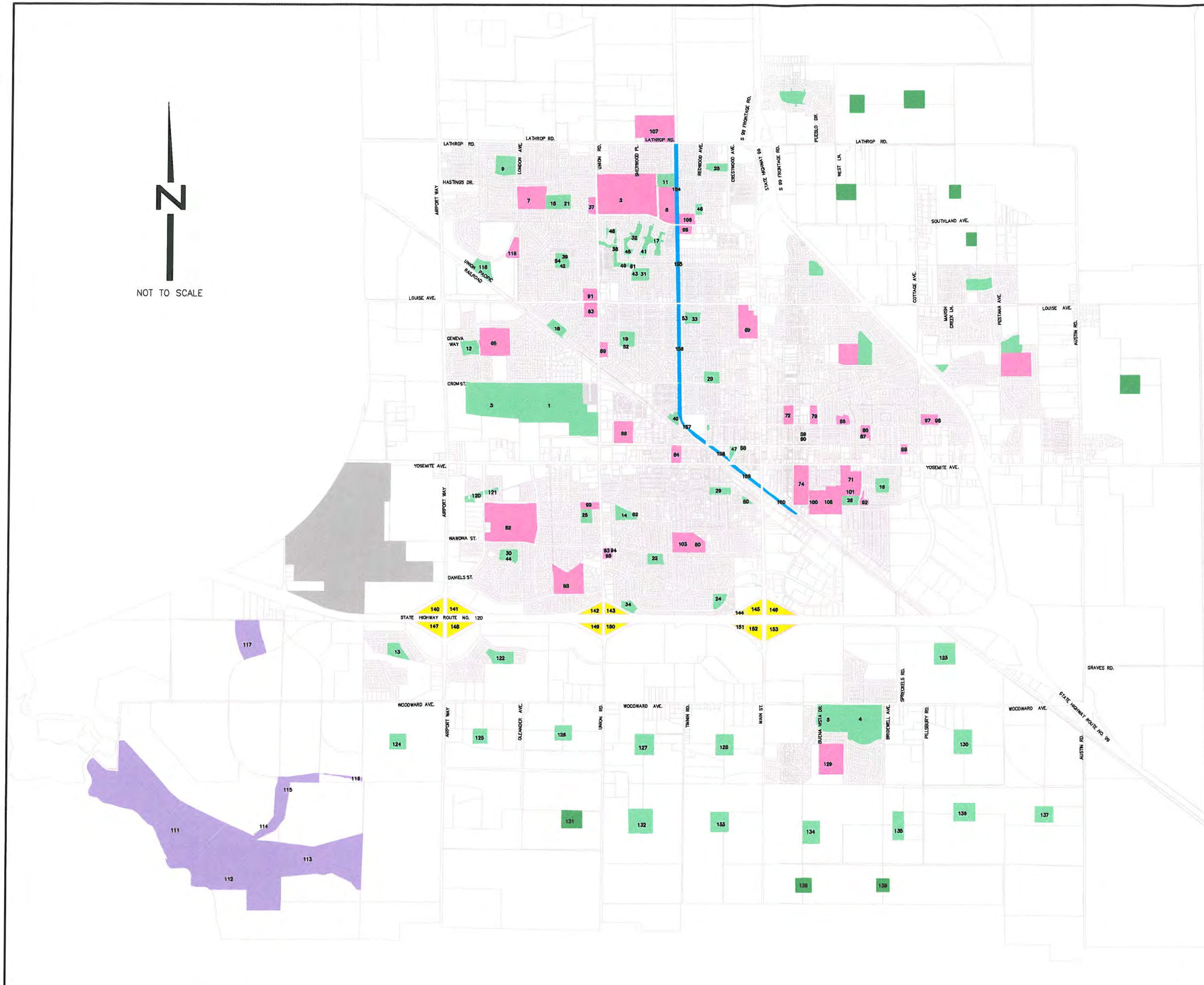
The 1996 investigation also evaluated the projected recycled water quality from the Manteca WQCF in terms of its suitability for irrigation. Considering multiple water quality parameters, it was concluded that the recycled water would be suitable for landscape irrigation without restrictions.

**Current Recycled Water Demands**

For this memorandum, water reuse sites have been identified using the General Plan [4] land use diagram and by conducting a windshield survey. Parcels zoned for parks, schools, churches and open space within the City were identified. Additionally, sites identified in previous water recycling evaluations were re-visited and the area along Tidewater Bikeway and Caltrans right-of-ways along SR-120 were included. In total, one hundred sixty possible water recycling sites were located (see Figure 1). However, to be economically feasible, reuse sites should be either close to the WQCF and/or clustered near other sites. Therefore, from the one hundred sixty potential sites, twenty-six sites were eliminated from further consideration on the basis of being either too far or too isolated from other locations. The sites eliminated from consideration were principally located to the east of State Route 99 (SR-99). The remaining one hundred thirty-four reuse sites represent a total area of 1,285 acres, of which 817 acres are assumed to be irrigable. Caltrans right-of-ways along SR-120 accounts for approximately 65 acres and the Tidewater Bikeway accounts for approximately 26 acres. Most of the future water recycling demands are located south of SR-120. The irrigable area for each site was estimated based on the following assumptions:

1. Parks - 90 percent of area is irrigable
2. Schools – 50 percent of area is irrigable
3. Churches – 50 percent of area is irrigable
4. Open Space – 50 percent of area is irrigable

The gross area and projected irrigable area for each reuse site are presented in Table 1.




LEGEND

| | |
|---|---|
| | P (PARK) |
| | PQP (PUBLIC/QUASI-PUBLIC) |
| | OS (OPEN SPACE) |
| | UR-P |
| | TIDEWATER BIKEWAY |
| | CALTRANS RIGHT-OF-WAY |
| | WASTEWATER QUALITY CONTROL FACILITY |
| 23 | REUSE SITE NUMBER 23. REFER TO TABLE 1 FOR ADDITIONAL INFORMATION |

NOTES:

1. PARCELS WITHOUT A NUMBER ARE NOT RECOMMENDED FOR REUSE SITES DUE TO LOCATION AND/OR SIZE

**FIGURE 1**

CITY OF MANTECA
WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE

UPDATE OF URBAN WATER
RECYCLING OPPORTUNITIES

**LOCATION OF POTENTIAL WATER REUSE SITES**

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N:\SA1224\05\Cadd\Reuse sites.dwg KMP 09/13/04

TABLE I
CITY OF MANTECA
WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE
UPDATE OF URBAN WATER RECYCLING OPPORTUNITIES
SUMMARY OF RECYCLED WATER DEMANDS AT GENERAL PLAN BUILD-OUT

| Reuse Site No. | Land Use Designation | Description | Total Area, acres | Irrigable area[b], acres | Reclamation Demands: Applied Water[c], ac-ft/yr | Reclamation Demands: Max. day demand[d], | Reclamation Demands: Peak hour delivery[e], |
|---|---|---|---|---|---|---|---|
| 1 | Park | Manteca Park Golf Course | 75.39 | 67.85 | 305.3 | 0.655 | 455 |
| 2 | Public/Quasi-Public | East Union School | 57.08 | 28.54 | 128.4 | 0.276 | 574 |
| 3 | Park | Manteca Park Golf Course | 44.20 | 39.78 | 179.0 | 0.384 | 267 |
| 4 | Park | Woodward Community Park | 33.43 | 30.09 | 135.4 | 0.291 | 605 |
| 5 | Park | Woodward Community Park | 17.20 | 15.48 | 69.7 | 0.150 | 311 |
| 6[a] | | | | | | | |
| 7 | Public/Quasi-Public | George McParland School | 15.31 | 7.66 | 34.4 | 0.074 | 154 |
| 8 | Public/Quasi-Public | Neil Hafley School | 14.12 | 7.06 | 31.8 | 0.068 | 142 |
| 9 | Park | Chadwick Square | 9.00 | 8.10 | 36.5 | 0.078 | 163 |
| 10 | | | | | | | |
| 11 | Park | Northgate Park | 5.45 | 4.91 | 22.1 | 0.047 | 99 |
| 12 | Park | Villa Ticino Park | 5.59 | 5.03 | 22.6 | 0.049 | 101 |
| 13 | Park | Bella Vista Park | 5.02 | 4.52 | 20.3 | 0.044 | 91 |
| 14 | Park | Yosemite Village Park | 4.89 | 4.40 | 19.8 | 0.043 | 89 |
| 15 | Park | Doxey Park | 4.45 | 4.01 | 18.0 | 0.039 | 81 |
| 16 | Park | Curran Grove | 4.42 | 3.98 | 17.9 | 0.038 | 80 |
| 17 | Park | Park West | 4.34 | 3.91 | 17.6 | 0.038 | 79 |
| 18 | Park | Mayor's Park | 3.99 | 3.59 | 16.2 | 0.035 | 72 |
| 19 | Park | Greystone Park | 4.00 | 3.60 | 16.2 | 0.035 | 72 |
| 20 | Park | Bay Meadows Park | 4.00 | 3.60 | 16.2 | 0.035 | 72 |
| 21 | Park | Doxey Park | 3.99 | 3.59 | 16.2 | 0.035 | 72 |
| 22 | Park | Sequoia Park | 3.98 | 3.58 | 16.1 | 0.035 | 72 |
| 23 | Park | Crestwood Park | 3.85 | 3.47 | 15.6 | 0.033 | 70 |
| 24 | Park | Cotta Park | 3.71 | 3.34 | 15.0 | 0.032 | 67 |
| 25 | Park | Union West Park | 3.69 | 3.32 | 14.9 | 0.032 | 67 |
| 26 | | | | | | | |
| 27 | | | | | | | |
| 28 | Park | Lincoln Park | 3.67 | 3.30 | 14.9 | 0.032 | 66 |
| 29 | Park | Southside Park | 3.31 | 2.98 | 13.4 | 0.029 | 60 |
| 30 | Park | Roberts Park | 3.29 | 2.96 | 13.3 | 0.029 | 60 |
| 31 | Park | Colony Park | 3.26 | 2.93 | 13.2 | 0.028 | 59 |
| 32 | Park | Park West | 3.12 | 2.81 | 12.6 | 0.027 | 56 |
| 33 | Park | Franciscan Park | 2.61 | 2.35 | 10.6 | 0.023 | 47 |
| 34 | Park | Quail Ridge Park | 3.05 | 2.75 | 12.4 | 0.027 | 55 |
| 35 | | | | | | | |
| 36 | | | | | | | |
| 37 | Public/Quasi-Public | Church of Jesus Christ of Latter Day Saints | 2.66 | 1.33 | 6.0 | 0.013 | 27 |
| 38 | Park | Park West | 2.36 | 2.12 | 9.6 | 0.021 | 43 |
| 39 | Park | St. Francis Park | 2.14 | 1.93 | 8.7 | 0.019 | 39 |
| 40 | Park | Walnut Place Park | 2.02 | 1.82 | 8.2 | 0.018 | 37 |
| 41 | Park | Park West | 1.80 | 1.62 | 7.3 | 0.016 | 33 |
| 42 | Park | St. Francis Park | 1.78 | 1.60 | 7.2 | 0.015 | 32 |
| 43 | Park | Colony Park | 1.76 | 1.58 | 7.1 | 0.015 | 32 |
| 44 | Park | Roberts Park | 1.78 | 1.60 | 7.2 | 0.015 | 32 |
| 45 | Park | Park West | 1.69 | 1.52 | 6.8 | 0.015 | 31 |
| 46 | Park | William Martin Park | 1.56 | 1.40 | 6.3 | 0.014 | 28 |
| 47 | Park | Library Park | 1.47 | 1.32 | 6.0 | 0.013 | 27 |
| 48 | Park | Park West | 1.40 | 1.26 | 5.7 | 0.012 | 25 |
| 49 | Park | Park West | 1.38 | 1.24 | 5.6 | 0.012 | 25 |
| 50 | Park | Baccilieri Park | 1.27 | 1.14 | 5.1 | 0.011 | 23 |
| 51 | | | | | | | |
| 52 | Park | Greystone Park | 1.20 | 1.08 | 4.9 | 0.010 | 22 |
| 53 | Park | Franciscan Park | 0.95 | 0.86 | 3.8 | 0.008 | 17 |
| 54 | Park | St. Francis Park | 0.85 | 0.77 | 3.4 | 0.007 | 15 |
| 55 | | | | | | | |
| 56 | | | | | | | |
| 57 | | | | | | | |
| 58 | Park | Wilson Park | 0.38 | 0.34 | 1.5 | 0.003 | [illegible] |
| 59 | Park | Hilderband Park | 0.29 | 0.26 | 1.2 | 0.003 | 5 |
| 60 | Park | Hilderband Park | 0.29 | 0.25 | 1.2 | 0.003 | 5 |

TABLE 1
CITY OF MANTECA
WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE
UPDATE OF URBAN WATER RECYCLING OPPORTUNITIES
SUMMARY OF RECYCLED WATER DEMANDS AT GENERAL PLAN BUILD-OUT

| Reuse Site No. | Land Use Designation | Description | Total Area, acres | Irrigable area[b], acres | Reclamation Demands: Applied Water[c], ac-ft/yr | Max. day demand[d] | Peak hour delivery[e] |
|---|---|---|---|---|---|---|---|
| 61 | Park | Park West | 0.18 | 0.16 | 0.7 | 0.002 | 3 |
| 62 | Park | Yosemite Village Park | 0.10 | 0.09 | 0.4 | 0.001 | 2 |
| 63 | | | | | | | |
| 64 | | | | | | | |
| 65 | Public/Quasi-Public | Stella Brockman School | 19.52 | 9.76 | 43.9 | 0.094 | 196 |
| 66 | Public/Quasi-Public | Senior Center | 9.52 | 4.76 | 21.4 | 0.046 | 96 |
| 67 | | | | | | | |
| 68 | Public/Quasi-Public | Brock Elliott School | 18.43 | 9.22 | 41.5 | 0.089 | 185 |
| 69 | Public/Quasi-Public | Golden West School | 13.41 | 6.71 | 30.2 | 0.065 | 135 |
| 70 | | | | | | | |
| 71 | Public/Quasi-Public | Lincoln Elementary School | 9.95 | 4.98 | 22.4 | 0.048 | 100 |
| 72 | Public/Quasi-Public | Manteca Adult School | 4.14 | 2.07 | 9.3 | 0.020 | 42 |
| 73 | | | | | | | |
| 74 | Public/Quasi-Public | Manteca High School | 13.64 | 6.82 | 30.7 | 0.066 | 137 |
| 75 | | | | | | | |
| 76 | | | | | | | |
| 77 | | | | | | | |
| 78 | | | | | | | |
| 79 | Public/Quasi-Public | St. Anthony's Catholic Church | 3.06 | 1.53 | 6.9 | 0.015 | 31 |
| 80 | Public/Quasi-Public | Sequoia High | 5.33 | 2.67 | 12.0 | 0.026 | 54 |
| 81 | | | | | | | |
| 82 | Public/Quasi-Public | Sierra High School | 46.22 | 23.11 | 104.0 | 0.223 | 465 |
| 83 | Public/Quasi-Public | Union Cemetary | 4.40 | 2.20 | 9.9 | 0.021 | 44 |
| 84 | Public/Quasi-Public | Sequoia Annex School | 3.78 | 1.89 | 8.5 | 0.018 | 38 |
| 85 | Public/Quasi-Public | 1st Baptist Church | 2.46 | 1.23 | 5.5 | 0.012 | 25 |
| 86 | Public/Quasi-Public | St. Paul's United Methodist Church | 2.45 | 1.23 | 5.5 | 0.012 | 25 |
| 87 | Public/Quasi-Public | St. Paul's United Methodist Church | 0.43 | 0.22 | 1.0 | 0.002 | 4 |
| 88 | Public/Quasi-Public | Fellowship Baptist Church of Manteca | 1.38 | 0.69 | 3.1 | 0.007 | 14 |
| 89 | Public/Quasi-Public | Freedom Christian Center | 2.55 | 1.28 | 5.7 | 0.012 | 26 |
| 90 | | | | | | | |
| 91 | Public/Quasi-Public | 1st Christian Church | 3.81 | 1.91 | 8.6 | 0.018 | 38 |
| 92 | Public/Quasi-Public | Manteca Fire Station | 1.32 | 0.66 | 3.0 | 0.006 | 13 |
| 93 | Public/Quasi-Public | Sequoia Heights Baptist Church | 0.74 | 0.37 | 1.7 | 0.004 | 7 |
| 94 | Public/Quasi-Public | Sequoia Heights Baptist Church | 0.24 | 0.12 | 0.5 | 0.001 | 2 |
| 95 | Public/Quasi-Public | Sequoia Heights Baptist Church | 1.23 | 0.62 | 2.8 | 0.006 | 12 |
| 96 | Public/Quasi-Public | Doctors Hospital of Manteca | 0.95 | 0.48 | 2.1 | 0.005 | 10 |
| 97 | Public/Quasi-Public | Doctors Hospital of Manteca | 2.92 | 1.46 | 6.6 | 0.014 | 29 |
| 98 | Public/Quasi-Public | (Northgate Community) Church | 2.37 | 1.19 | 5.3 | 0.011 | 24 |
| 99 | Public/Quasi-Public | Manteca Adventist School | 3.02 | 1.51 | 6.8 | 0.015 | 30 |
| 100 | Public/Quasi-Public | Manteca High | 4.43 | 2.22 | 10.0 | 0.021 | 45 |
| 101 | Public/Quasi-Public | Lincoln Elementary School | 3.43 | 1.72 | 7.7 | 0.017 | 35 |
| 102 | | | | | | | |
| 103 | Public/Quasi-Public | Sequoia High | 8.96 | 4.48 | 20.2 | 0.043 | 90 |
| 104 | | | | | | | |
| 105 | | | | | | | |
| 106 | Public/Quasi-Public | United Lutheran Church | 3.84 | 1.92 | 8.6 | 0.019 | 39 |
| 107 | Public/Quasi-Public | Calvary Community Church | 23.98 | 11.99 | 54.0 | 0.116 | 241 |
| 108 | Public/Quasi-Public | Manteca High | 15.35 | 7.68 | 34.5 | 0.074 | 154 |
| 109 | | | | | | | |
| 110 | | | | | | | |
| 111 | Open Space | | 204.43 | 102.22 | 460.0 | 0.987 | 2057 |
| 112 | Open Space | | 85.27 | 42.64 | 191.9 | 0.412 | 858 |
| 113 | Open Space | | 71.37 | 35.69 | 160.6 | 0.345 | 718 |
| 114 | Open Space | | 9.23 | 4.62 | 20.8 | 0.045 | 93 |
| 115 | Open Space | | 14.29 | 7.15 | 32.2 | 0.069 | 144 |
| 116 | Open Space | | 1.05 | 0.53 | 2.4 | 0.005 | 11 |
| 117 | Open Space | | 21.84 | 10.92 | 49.1 | 0.105 | 220 |
| 118 | Park | Primavera Park | 6.98 | 6.28 | 28.3 | 0.061 | 126 |
| 119 | Public/Quasi-Public | George McParland Annex School | 4.17 | 2.09 | 9.4 | 0.020 | 42 |
| 120 | Park | Gonsalves Park | [illegible] | 1.95 | 8.8 | 0.019 | 39 |

TABLE 1
CITY OF MANTECA
WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE
UPDATE OF URBAN WATER RECYCLING OPPORTUNITIES
SUMMARY OF RECYCLED WATER DEMANDS AT GENERAL PLAN BUILD-OUT

| Reuse Site No. | Land Use Designation | Description | Total Area, acres | Irrigable area[b], acres | Reclamation Demands: Applied Water[c], ac-ft/yr | Reclamation Demands: Max. day demand[d] | Reclamation Demands: Peak hour delivery[e] |
|---|---|---|---|---|---|---|---|
| 121 | Park | Gonsalves Estates Park | 1.13 | 1.02 | 4.6 | 0.010 | 20 |
| 122 | Park | Dutra Farms S.E. Park | 6.95 | 6.26 | 28.1 | 0.060 | 126 |
| 123 | Park | | 10.49 | 9.44 | 42.5 | 0.091 | 190 |
| 124 | Park | | 5.67 | 5.10 | 23.0 | 0.049 | 103 |
| 125 | Park | | 4.72 | 4.25 | 19.1 | 0.041 | 85 |
| 126 | Park | | 5.73 | 5.16 | 23.2 | 0.050 | 104 |
| 127 | Park | | 9.21 | 8.29 | 37.3 | 0.080 | 167 |
| 128 | Park | | 8.69 | 7.82 | 35.2 | 0.076 | 157 |
| 129 | Public/Quasi-Public | Future School | 17.65 | 8.83 | 39.7 | 0.085 | 178 |
| 130 | Park | | 10.25 | 9.23 | 41.5 | 0.089 | 186 |
| 131 | Park | | 8.46 | 7.61 | 34.3 | 0.074 | 153 |
| 132 | Park | | 13.65 | 12.29 | 55.3 | 0.119 | 247 |
| 133 | Park | | 8.27 | 7.44 | 33.5 | 0.072 | 150 |
| 134 | Park | | 9.03 | 8.13 | 36.6 | 0.078 | 164 |
| 135 | Park | | 7.16 | 6.44 | 29.0 | 0.062 | 130 |
| 136 | Park | | 9.06 | 8.15 | 36.7 | 0.079 | 164 |
| 137 | Park | | 6.86 | 6.17 | 27.8 | 0.060 | 124 |
| 138 | Park | | 5.32 | 4.79 | 21.5 | 0.046 | 96 |
| 139 | Park | | 3.98 | 3.58 | 16.1 | 0.035 | 72 |
| 140 | Open Space | CalTrans Right-of-Way | 6.00 | 3.00 | 13.5 | 0.029 | 60 |
| 141 | Open Space | CalTrans Right-of-Way | 6.00 | 3.00 | 13.5 | 0.029 | 60 |
| 142 | Open Space | CalTrans Right-of-Way | 3.95 | 1.98 | 8.9 | 0.019 | 40 |
| 143 | Open Space | CalTrans Right-of-Way | 3.95 | 1.98 | 8.9 | 0.019 | 40 |
| 144 | Open Space | CalTrans Right-of-Way | 1.12 | 0.56 | 2.5 | 0.005 | 11 |
| 145 | Open Space | CalTrans Right-of-Way | 4.96 | 2.48 | 11.2 | 0.024 | 50 |
| 146 | Open Space | CalTrans Right-of-Way | 6.75 | 3.37 | 15.2 | 0.033 | 68 |
| 147 | Open Space | CalTrans Right-of-Way | 6.00 | 3.00 | 13.5 | 0.029 | 60 |
| 148 | Open Space | CalTrans Right-of-Way | 6.00 | 3.00 | 13.5 | 0.029 | 60 |
| 149 | Open Space | CalTrans Right-of-Way | 3.95 | 1.98 | 8.9 | 0.019 | 40 |
| 150 | Open Space | CalTrans Right-of-Way | 3.95 | 1.98 | 8.9 | 0.019 | 40 |
| 151 | Open Space | CalTrans Right-of-Way | 1.12 | 0.56 | 2.5 | 0.005 | 11 |
| 152 | Open Space | CalTrans Right-of-Way | 4.96 | 2.48 | 11.2 | 0.024 | 50 |
| 153 | Open Space | CalTrans Right-of-Way | 6.75 | 3.37 | 15.2 | 0.033 | 68 |
| 154 | Open Space | Tidewater Biketrail | 5.98 | 2.99 | 13.4 | 0.029 | 60 |
| 155 | Open Space | Tidewater Biketrail | 5.84 | 2.92 | 13.1 | 0.028 | 59 |
| 156 | Open Space | Tidewater Biketrail | 5.86 | 2.93 | 13.2 | 0.028 | 59 |
| 157 | Open Space | Tidewater Biketrail | 4.12 | 2.06 | 9.3 | 0.020 | 41 |
| 158 | Open Space | Tidewater Biketrail | 0.98 | 0.49 | 2.2 | 0.005 | 10 |
| 159 | Open Space | Tidewater Biketrail | 1.87 | 0.94 | 4.2 | 0.009 | 19 |
| 160 | Open Space | Tidewater Biketrail | 1.54 | 0.77 | 3.5 | 0.007 | 15 |
| | | Total | 1284.7 | 817.3 | 3677.8 | 7.9 | 15000.1 |

[a]Non-designated land use sites are not recommended for recycled water service due to location and/or size

[b]90% of total area for parks and golf course and 50% of total area for schools, churches, hospitals, cemetary, fire station, senior center and open space

[c]Based on a 4.5-feet per year application rate

[d]Based on a 0.355 inches per day maximum daily demand (in July)

[e]Assuming an 8-hour irrigation period, except for the 24-hour irrigation period for the Manteca Golf Course

Table 1 also identifies the annual, maximum daily, and peak hourly recycled water demand for each site. The total recycled water demand for the one hundred thirty-four reuse sites at General Plan build-out is approximately 3,700 ac-ft/yr (recycled water demand for existing land uses is 1,750 ac-ft/yr). During July, the reuse sites are projected to require a daily recycled water delivery of 7.9 mgd. The peak hour delivery would be approximately 15,000 gpm.

The water applied to each site was estimated by multiplying the site's irrigated acreage by the depth of water required. The annual demand, as indicated in Table 2, was estimated to be approximately 54-in/yr. The maximum daily demand of 0.35-in/day was estimated based on the highest monthly (July) irrigation requirement of 11.0-in/month divided by 31 days. The peak hourly demand, used to size the recycled water distribution system, was based on an irrigation cycle for parks, schools, and the cemetery from 10 p. m. to 6 a. m., a period of eight hours per day. This restricted irrigation window reduces the potential for public contact with recycled water. The golf course was assumed to receive water twenty-four hours per day. The delivery time for these potential customers is longer because it is assumed that ponds for temporary storage of irrigation water would be used at these sites. The peak delivery requirement occurs during the eight-hour period used for irrigation by all customers.

TABLE 2
CITY OF MANTECA
WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE
UPDATE OF URBAN WATER RECYCLING OPPORTUNITIES
LANDSCAPE IRRIGATION WATER BALANCE

| Month | ET[a] Evapotranspiration, in | $P_{10}$[b] Precipitation, in | CD[c] Crop Demand, in | LR[d] Leaching Req't, in | AW[e] Applied Water, in |
|---|---|---|---|---|---|
| Jan | 0.92 | 3.12 | - | - | - |
| Feb | 1.49 | 2.70 | - | - | - |
| Mar | 3.05 | 2.42 | 0.63 | 0.06 | 0.9 |
| Apr | 4.72 | 1.35 | 3.37 | 0.34 | 4.6 |
| May | 6.35 | 0.52 | 5.83 | 0.58 | 8.0 |
| Jun | 7.68 | 0.11 | 7.57 | 0.76 | 10.4 |
| Jul | 8.06 | 0.08 | 7.98 | 0.80 | 11.0 |
| Aug | 6.83 | 0.07 | 6.76 | 0.68 | 9.3 |
| Sep | 5.31 | 0.25 | 5.06 | 0.51 | 7.0 |
| Oct | 3.30 | 0.91 | 2.39 | 0.24 | 3.3 |
| Nov | 1.42 | 2.07 | - | - | - |
| Dec | 0.61 | 2.88 | - | - | - |
| Total | 49.74 | 16.48 | 39.59 | 3.96 | 54.4 |

[a] $ET_0$: Evapotranspiration rates computed from Figure 5-11, *Irrigation with Reclaimed Municipal Wastewater* [5].

[b] $P_{10}$: One in ten year precipitation values from *California Rainfall Summary*, Manteca gage station [6].

[c] CD: Crop Demand = $(ET-P_{10})$ positive values.

[d] LR: Leaching Requirement = 0.10*(CD).

[e] AW: Applied Water = (CD+LR)/0.80.

## Recycled Water System Facilities

In this section, recycled water storage, pumping, and distribution system facilities are described.

Storage System

At General Plan build-out, the average dry weather flow to the WQCF is 27 mgd which is greater that the projected recycled water delivery requirement of 15,000 gpm. The daily recycled water demand schedule will require some operational storage, however. Typical operational storage requirements are approximately one-half of the maximum daily demand or 4.0 MG (50 percent of 7.9 mgd).

Delivery (Pumping and Conveyance)

A recycled water pump station has been sized to meet peak delivery schedules with the largest pump removed from service. A delivery pressure of 60 psi is assumed for landscape irrigation customers except where onsite storage ponds are available such as at the golf course. Pipe sizes for the recycled water distribution system were selected based on a criterion of headloss no greater than 8 feet per 1,000 feet, using a Hazen-Williams coefficient of 120. Peak hour deliveries to each potential irrigation site were used to size the pipelines. The layout of the distribution system is presented in Figure 2.

It can be seen in Figure 2 that the recycled water distribution system consists of a large loop encircling the majority of the City with one branch line to the north and a second branch line to the south. The recycled water pipeline has been routed to provide access to the majority of the reuse sites. Use of a looped system improves pressure distribution and provides redundancy in the event of a pipe failure. The existing 12-inch Stage II force main on Woodward Road could be converted to a 12-inch recycled water pipeline if the timing is appropriate.

The head losses associated with flow through each pipe were used to determine the overall system head loss and pumping requirement. The greatest headloss (45 psi) occurs along the route from the WQCF to the eastern sites on Woodward Road. Assuming a minimum endpoint delivery pressure of 60 psi, the initial pressure at the WQCF is approximately 105 psi. To maintain a pumping pressure of 105 psi at 15,000 gpm requires approximately 654 horsepower (assuming pump efficiency of 75 percent). Proposed design criteria for the recycled water delivery system are presented in Table 3.


LEGEND

| | |
|---|---|
| —— | PROPOSED PIPELINE |
| 24" | PROPOSED PIPE SIZE |
| ▒ | WASTEWATER QUALITY CONTROL FACILITY |

# FIGURE 2

CITY OF MANTECA
WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE

UPDATE OF URBAN WATER RECYCLING OPPORTUNITIES

**PROPOSED RECYCLED WATER DISTRIBUTION PIPELINES**

NOLTE
BEYOND ENGINEERING

N:\SA1224\06\Cadd\Reuse sites.dwg KMP 09/13/04

TABLE 3
CITY OF MANTECA
WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE
UPDATE OF URBAN WATER RECYCLING OPPORTUNITIES
DESIGN CRITERIA FOR RECYCLED WATER DELIVERY SYSTEM

| Item | Units | Value |
|---|---|---|
| Recycled Water Pump Station | | |
| Peak delivery flow | gpm | 15,000 |
| Maximum operating head | psi | 105 |
| Total connected horsepower at build-out | hp | 654 |
| Recycled Water Distribution Pipe System | | |
| Discharge pressure | psi | 60 |
| Pipe material | type | plastic |
| Headloss | ft/1000 ft | 8 |
| Hazen-Williams coefficient | C | 120 |
| Maximum velocity (at peak flow) | ft/s | 5 |
| Minimum velocity (at peak flow) | ft/s | 3 |
| 30-in pipe | ft | 8,200 |
| 24-in pipe | ft | 19,200 |
| 20-in pipe | ft | 12,000 |
| 18-in pipe | ft | 6,200 |
| 16-in pipe | ft | 11,200 |
| 12-in pipe | ft | 12,200 |
| Total Length of pipe | ft | 69,000 |

## Summary

Based on the General Plan build-out land use, one hundred thirty-four sites within the City comprising 817 acres, have been identified as candidates for receiving recycled water. These sites could potentially use 3,700 ac-ft/yr of recycled water, which represents 33 percent of the annual Phase III wastewater flow of 11,230 ac-ft/yr (10 mgd). Treatment, storage, and distribution facilities would be necessary to meet this demand.

## REFERENCES

[1] *Evaluation of Agricultural Irrigation with Recycled Water*, prepared by Nolte Associates, July 2004.

[2] *Manteca Effluent to Land Disposal Study, Technical Memorandum No. 3-2, Evaluation of Urban Water Recycling Opportunities from the Manteca Wastewater Control Facility*, prepared by Nolte Associates, February 2002.

[3] *Water Reuse Investigation Preliminary Design Report, TM No.4 - Manteca Wastewater Quality Control Facility (WQCF) Phase III Expansion*, prepared by Nolte and Associates, January 1996.

[4] City of Manteca General Plan 2023 Policy Document, adopted October 6, 2003.

[5] *Irrigation with Reclaimed Municipal Wastewater – A Guidance Manual*, prepared by the California State Water Resources Control Board, July 1984.

[6] California Rainfall Summary, prepared by the California Department of Water Resources, 1981.

# APPENDIX C
# ROUTE STUDY FOR PARALLEL SEWER

On August 2nd, 2006 Nolte staff walked the existing outfall route between the WQCF and the discharge point at the San Joaquin River to identify potential constraints and pipeline route alignment opportunities for the design and construction of a future outfall. Existing utility maps and other information were collected from the City of Manteca and the San Joaquin County Surveyor. The alignment walk-through was divided into the following reaches: 1) beginning at the WQCF west towards McKinley Avenue; 2) from McKinley Avenue south to State Route 120 (SR-120); 3) from the SR-120 corridor between McKinley Avenue and the Union Pacific Railroad (UPRR); 4) UPRR corridor between SR-120 and the San Joaquin River. A brief summary of the potential constraints identified for each reach and a suggested alignment are provided below. A key map for the overall existing outfall is presented in Figure C-1.

a. Reach 1 – WQCF to McKinley Avenue

The walk-through began at the WQCF between the Secondary Effluent Storage Pond (SESP) and the Chemical Handling Building area. This reach ended at McKinley Avenue. Reach 1 is located on City property. Above ground utilities included overhead electrical transmission lines along the western portion of the reach. A significant number of buried pipelines are located between the SESP and Chemical Handling Building. The existing 36-inch pipeline runs underneath plant service roads and adjacent to pond berms. The existing outfall is approximately 3 feet below grade. Signs are located at several locations along the route. The existing outfall crosses a small irrigation ditch for approximately 30 feet before reaching McKinley Avenue. Site observations indicate that sufficient space is available throughout most of this reach for a future 36-inch outfall to be routed parallel to the existing outfall. A schematic representation of the existing utilities and constraints along Reach 1 is presented in Figure C-2.

b. Reach 2 –McKinley Avenue to SR-120

Reach 2 begins at the intersection of the WQCF property and is located along McKinley Avenue and ends south of SR-120. The existing outfall is located on the west side of McKinley Avenue, approximately ten feet from the road centerline with approximately 48 inches of cover. Existing utilities include overhead electrical and telephone lines on the east side of McKinley Avenue. Signage for a buried GTE cable was located on McKinley Avenue south of SR-120 where the pipeline turns west. The existing outfall is centered within a 20 foot permanent easement located within McKinley Avenue. The future outfall could be located east of the existing alignment, within the permanent easement. An additional 10 feet of temporary construction easement within private properties and Caltrans right-of-way (ROW) is recommend for this area. A schematic representation of the existing utilities and constraints along Reach 2 is presented in Figure C-3.

c. Reach 3 – SR-120 between McKinley Avenue and UPRR

Reach 3 parallels SR-120 from McKinley Avenue to the UPRR. The existing outfall is located along the northern perimeter of several private properties, and is outside of the Caltrans ROW for SR-120. The outfall is centered within a 20-foot permanent easement. No existing utilities were identified in the area. The future outfall could be located north or south of the existing alignment, within the permanent easement. An additional 10 feet of temporary construction easement is recommended for this area. A schematic representation of the existing utilities and constraints along Reach 3 is presented in Figure C-4.

d. <u>Reach 4 –UPRR between SR-120 and San Joaquin River</u>

Reach 4 parallels the UPRR from SR-120 to the San Joaquin River. Reach 4 is routed through the Oakwood Shores gated community, currently under construction. Site observations and a review of improvement plans for the community indicate that a 70 to 75 foot landscaped private access easement has been provided along the existing outfall alignment. Existing utilities in the area include overhead electrical lines, a 12-inch buried water line, and buried recycled water lines. Electrical vaults for landscaping, radio towers foundations, and a refueling tank with generator were also observed near the alignment. The outfall is located within a 20-foot permanent easement. The outfall is centered within this easement for a portion of the alignment. For the remainder of the alignment, the outfall is located five-feet from the southerly limit of the easement, to avoid an existing 12-inch water line.

The future outfall could be located south of the existing alignment. For sections of the alignment where the existing outfall is not centered within the permanent easement, an additional 10 feet of permanent easement may be required. A 10 foot temporary construction easement is recommended along the entire reach. The future outfall would probably be installed using trenchless methods when crossing below the east levee along the San Joaquin River. Therefore a 15 foot temporary construction easement is recommended for this area. Potential constraints for design and construction of the future outfall along this reach include avoiding mature oak trees near the river, access to the San Joaquin River, and access within the gated community. A schematic representation of the existing utilities and constraints along Reach 4 is presented in Figure C-5.

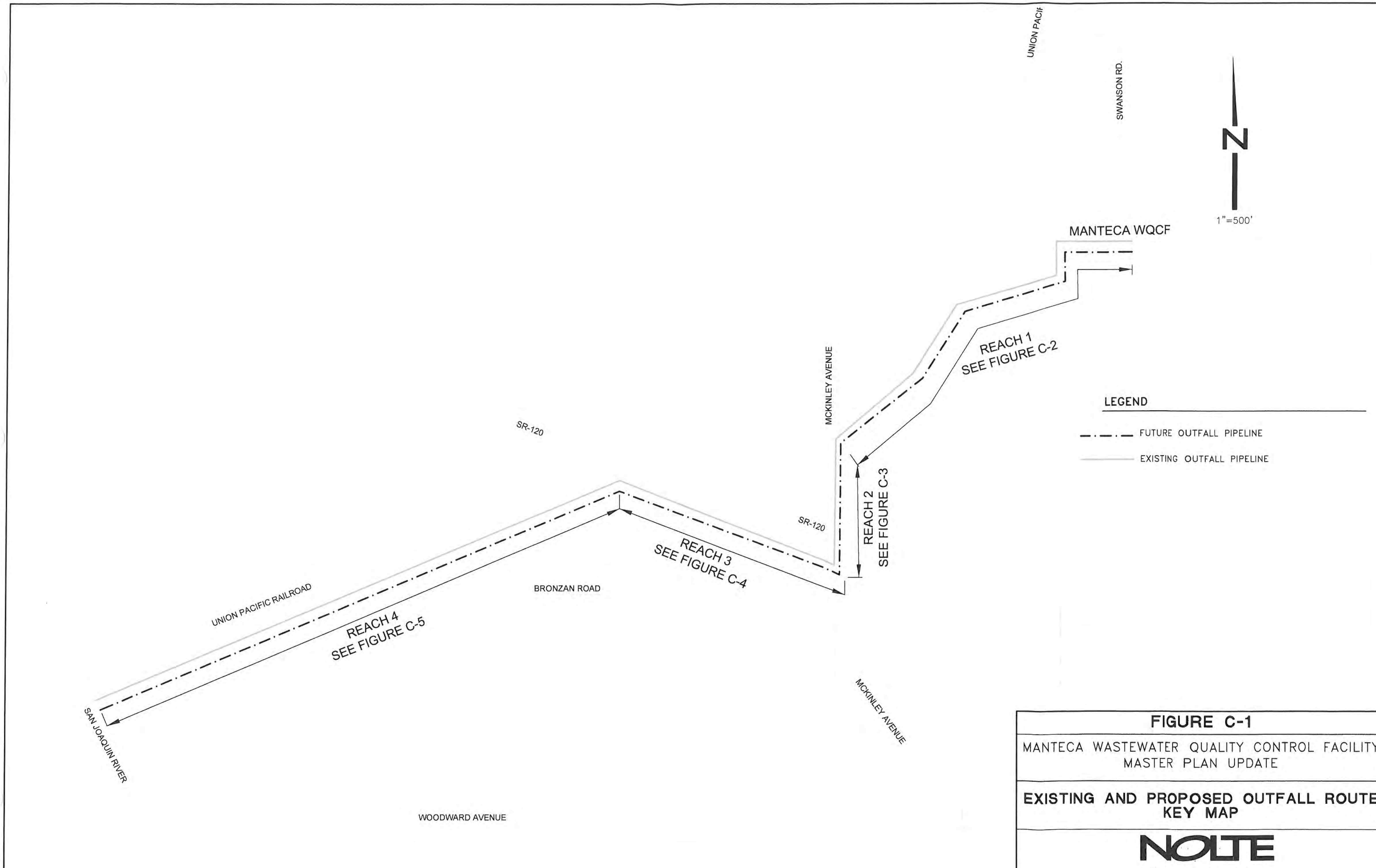


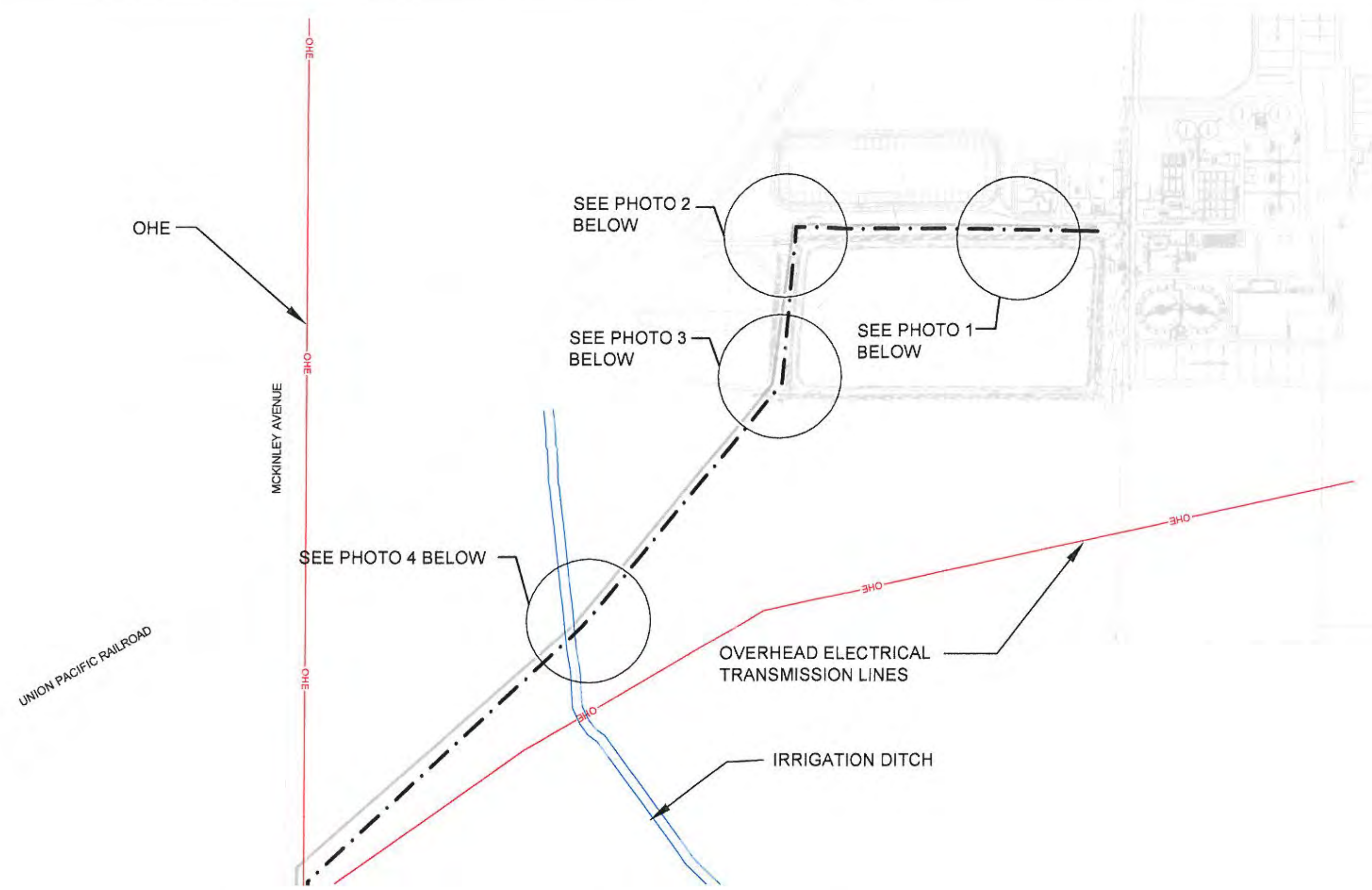

FIGURE C-1

MANTECA WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN UPDATE

EXISTING AND PROPOSED OUTFALL ROUTE KEY MAP

NOLTE
BEYOND ENGINEERING

N:\SA0122415\cadd\FIGURE C-1 EP 09/14/2006




N

1"=300'

# REACH 1– WQCF TO MCKINLEY AVENUE

## SCHEMATIC PLAN

NOT TO SCALE

**LEGEND**

- FUTURE OUTFALL PIPELINE
- EXISTING OUTFALL PIPELINE
- OVERHEAD ELECTRICAL AND TRANSMISSION LINES
- IRRIGATION DITCH

PHOTO 1

WQCF–FACING WEST
BETWEEN SECONDARY EFFLUENT
STORAGE POND AND CHEMICAL BUILDING

PHOTO 2

WQCF–FACING SOUTH
WESTERN LEVEE OF ECKERT POND

PHOTO 3

WQCF–FACING SOUTHWEST
SOUTH END OF ECKERT POND
NEAR APPLICATION FIELDS

PHOTO 4

WQCF–FACING WEST
AT IRRIGATION DITCH CROSSING

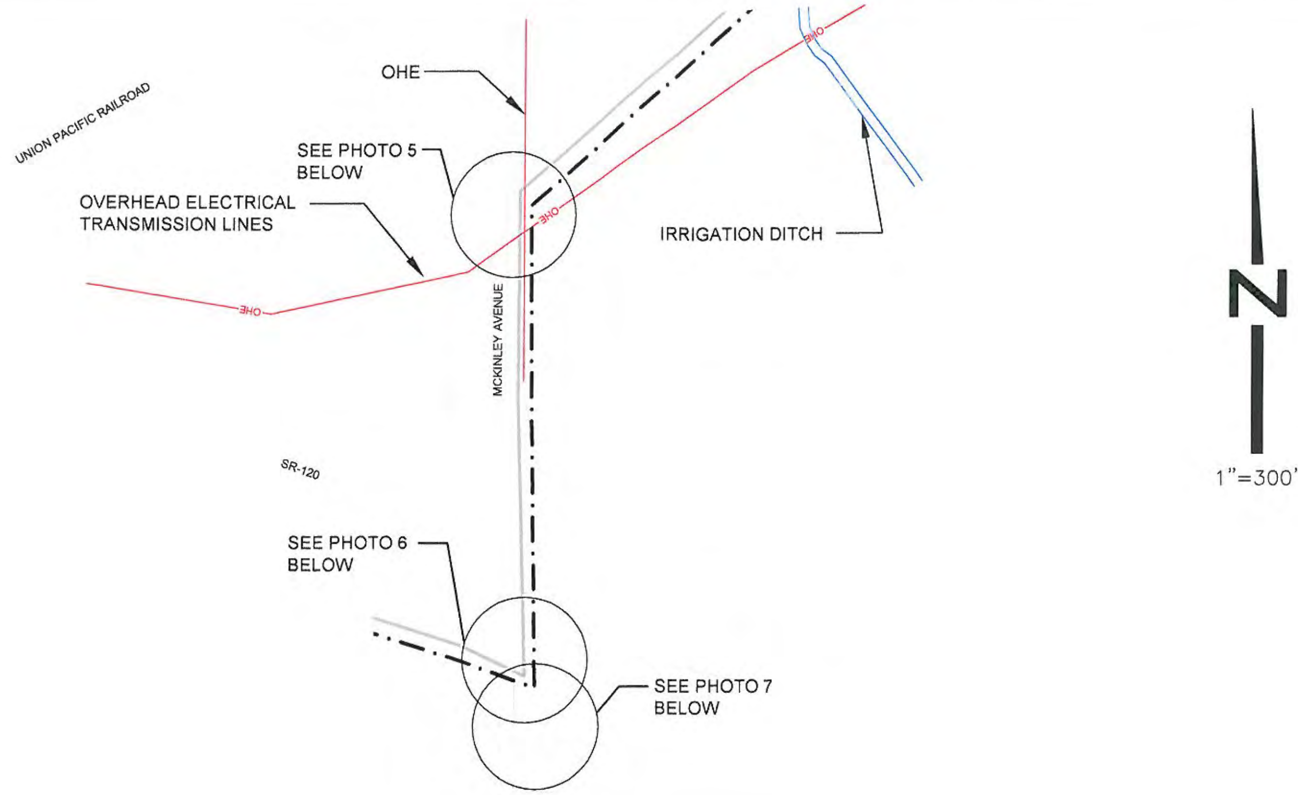
**FIGURE C-2**

MANTECA WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UNDATE

**REACH 1**
**WQCF TO MCKINLEY AVENUE**

NOLTE
BEYOND ENGINEERING

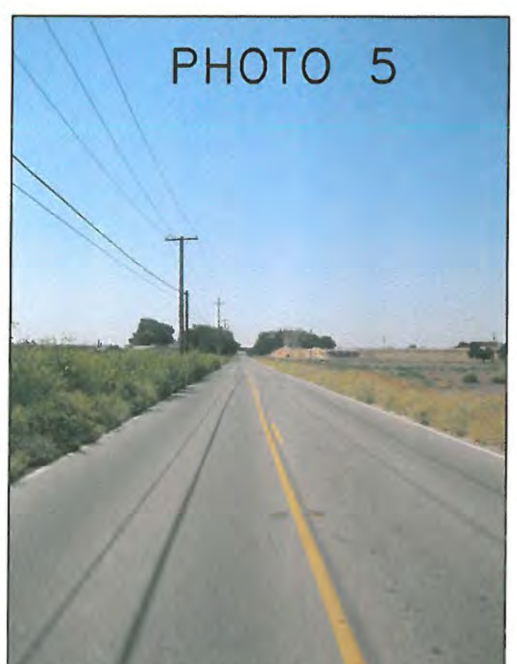
N:\SA0122415\cadd\FIGURE C-2 EP 08/15/2006



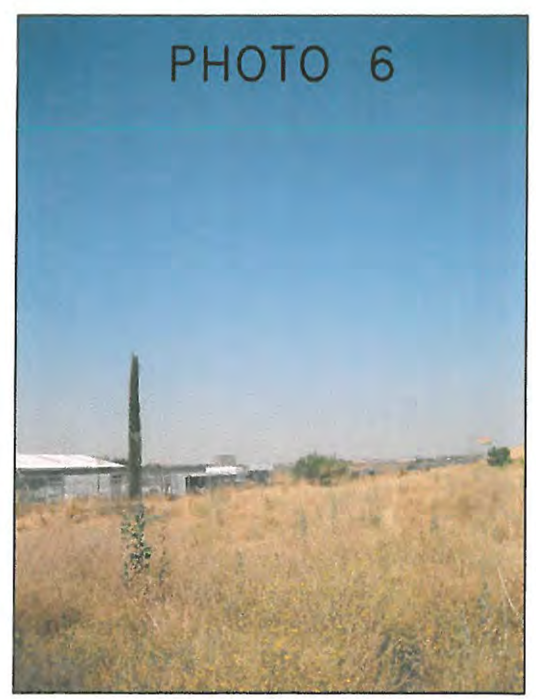

## REACH 2-MCKINLEY AVENUE TO SR-120

## SCHEMATIC PLAN

NOT TO SCALE




MCKINLEY AVENUE-FACING SOUTH
OVERHEAD UTILITIES ON EAST SIDE OF ROAD




MCKINLEY AVENUE-FACING WEST
BETWEEN BRONZAN ROAD AND SR-120


MCKINLEY AVENUE-FACING EAST
BURIED UTILITY CABLE SIGN FOR GTE

**LEGEND**

- FUTURE OUTFALL PIPELINE
- EXISTING OUTFALL PIPELINE
- OVERHEAD ELECTRICAL AND TRANSMISSION LINES
- IRRIGATION DITCH

**FIGURE C-3**

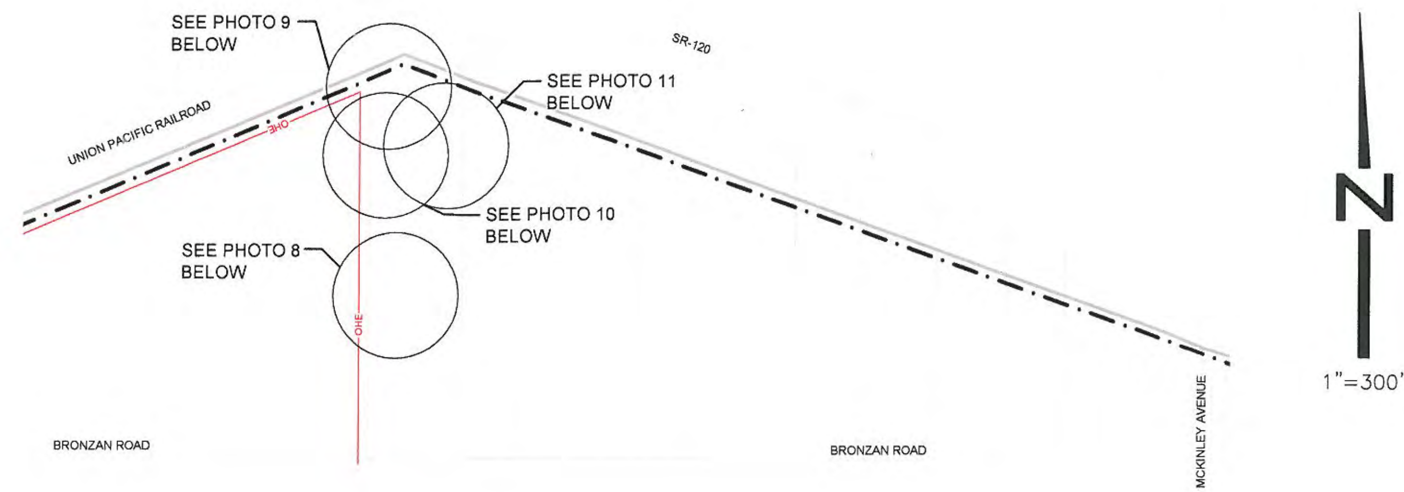
MANTECA WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE

**REACH 2**
**MCKINLEY AVENUE TO SR-120**

NOLTE
BEYOND ENGINEERING

N:\SA0122415\cadd\FIGURE C-3 EP 08/24/06


SERVICE ROAD FACING NORTH TO SR-120
AND UPRR INTERSECTION




## REACH 3- SR-120 BETWEEN MCKINLEY AVENUE AND UPRR

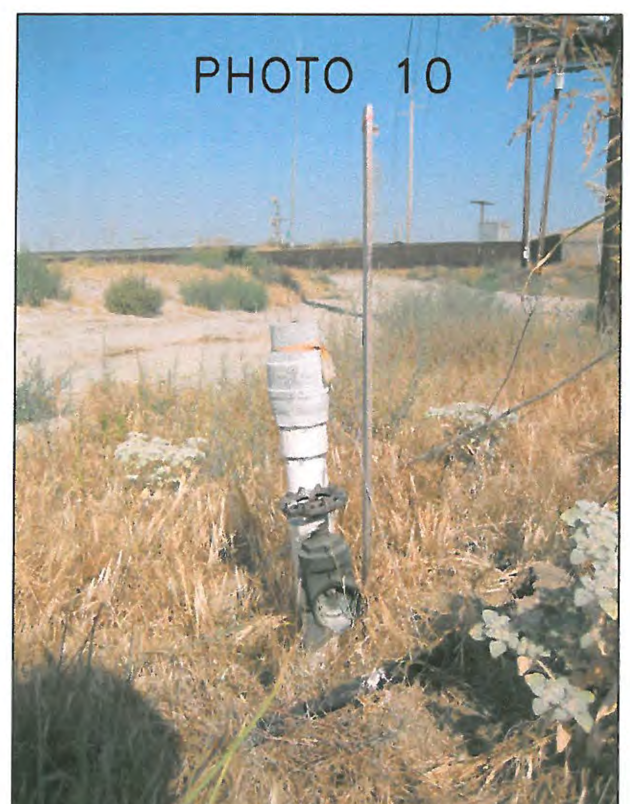
## SCHEMATIC PLAN

NOT TO SCALE

**LEGEND**

- FUTURE OUTFALL PIPELINE
- EXISTING OUTFALL PIPELINE
- OVERHEAD ELECTRICAL AND TRANSMISSION LINES


SERVICE ROAD FACING EAST TOWARD MCKINLEY AVENUE
BENCHMARKS BY ASSOCIATED ENGINEERING GROUP




SERVICE ROAD FACING WEST TOWARD MCKINLEY AVENUE
BLOW OFF VALVE ON EXISTING OUTFALL


SERVICE ROAD FACING EAST TO MCKINLEY AVENUE
OVERHEAD UTILITIES NORTH OF SERVICE ROAD

**FIGURE C-4**

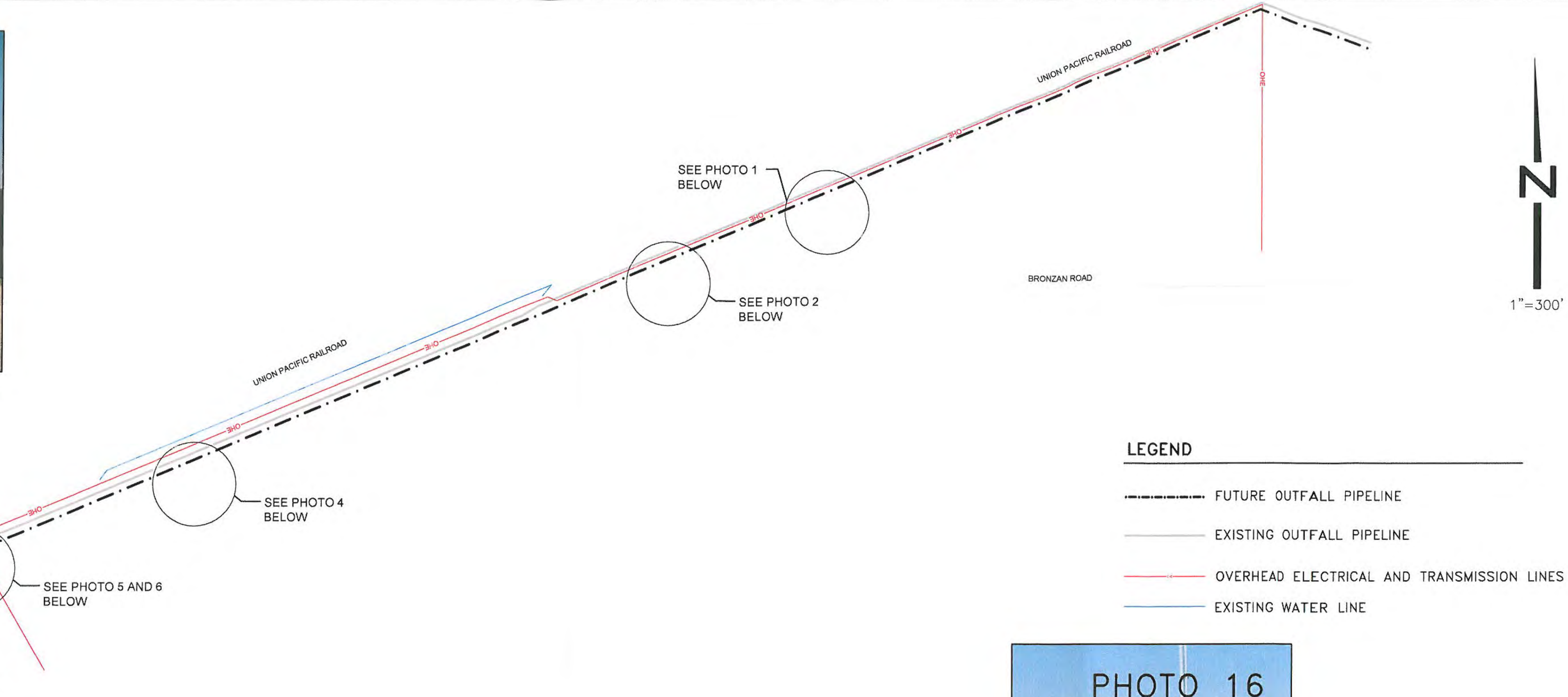
CITY OF MANTECA WASTEWATER QUALITY CONTROL FACILITY
MASTER PLAN UPDATE

**REACH 3**
**SR-120 BETWEEN MCKINLEY AVENUE**
**AND UPRR**

NOLTE
BEYOND ENGINEERING

N:\SA0122415\cadd\FIGURE C-4 EP 08/15/2006


SERVICE ROAD FACING WEST TOWARD SAN JOAQUIN RIVER
OVERHEAD UTILITIES NORTH OF SERVICE ROAD
RECYCLED WATER LINE SPRINKLER CONNECTIONS
SOUTH OF SERVICE ROAD



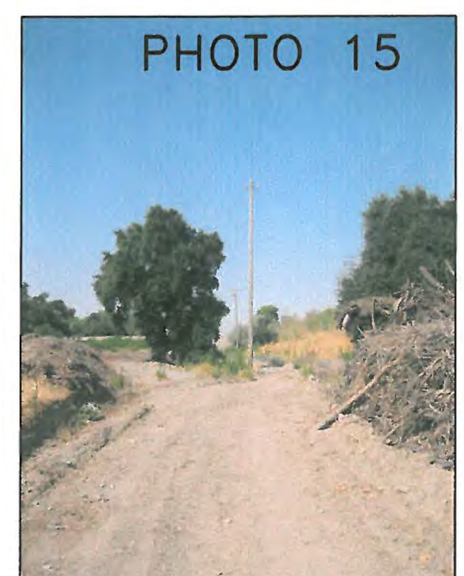

## REACH 4– UPRR TO SAN JOAQUIN RIVER

## SCHEMATIC PLAN

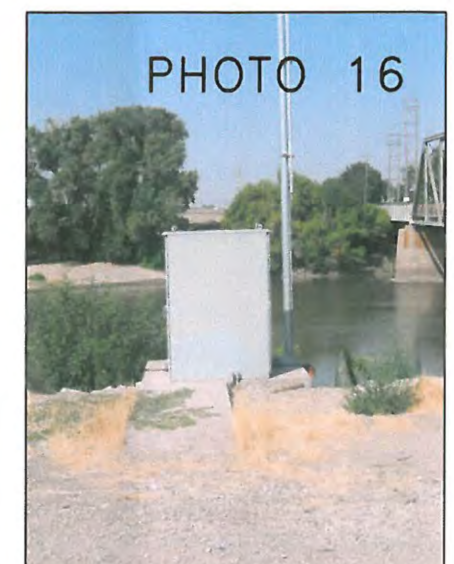
NOT TO SCALE


FACING SOUTH FROM SERVICE ROAD
RECLAIMED WATER SPRINKLER SYSTEM
APPROXIMATELY 30 FEET FROM OUTFALL PIPELINE


FACING WEST FROM SERVICE ROAD
FUEL TANK AND GENERATOR




SERVICE ROAD FACING SOUTHWEST
TO SAN JOAQUIN RIVER
NUMEROUS OAK TREES AROUND AREA




OUTFALL FACING SOUTHWEST FROM LEVEE
SAN JOAQUIN RIVER OUTFALL


END OF OUTFALL FACING EAST

| FIGURE C-5 |
|---|
| CITY OF MANTECA WASTEWATER QUALITY CONTROL FACILITY MASTER PLAN UPDATE |
| REACH 4<br>UPRR TO SAN JOAQUIN RIVER |
| NOLTE<br>BEYOND ENGINEERING |

# APPENDIX D
# UNIT COST INFORMATION FOR OPINION OF PROBABLE CONSTRUCTION COSTS

## Table D-1
## Unit Costs for Treatment Plant Expansion

| Unit Process | Unit | Unit Cost | Probable Construction Cost, $ |
|---|---|---|---|
| Influent Pump Station | MGD | 120,000 | 1,985,000 |
| Grit Removal/Primary Sedimentation | sq ft - primary Clarifier | 300 | 7,100,000 |
| Aeration Basins | MG | 1,560,400 | 16,072,000 |
| Secondary Clarification | ft - diameter | 10,900 | 8,357,000 |
| Tertiary Filtration | sq ft | 900 | 5,829,000 |
| UV Disinfection/Effluent Pumping | UV Lamp | 2,490 | 9,870,000 |
| Equalization Pond | MGD | 134,000 | 2,303,000 |
| Chemical Building | sq ft | 620 | 1,291,000 |
| Anaerobic Digestion/Sludge Heating | ft - diameter | 27,000 | 9,985,000 |
| Thickening Facilities | MGD | 166,000 | 2,840,000 |
| Dewatering Facilities | MGD | 65,000 | 1,119,000 |
| Odor Control | MGD | 100,000 | 1,659,000 |
| Subtotal | | | 68,410,000 |
| Civil/Site Improvements | Percent of Total | 5% | 3,421,000 |
| Yard Piping | Percent of Total | 10% | 6,841,000 |
| Electrical | Percent of Total | 30% | 20,523,000 |
| Subtotal | | | 99,195,000 |
| Outfall | Linear ft | 298.6 | 4,300,000 |
| Grand Total | | | 103,495,000 |

# APPENDIX E
# PROBABLE CONSTRUCTION COSTS FOR PARALLEL LAND OUTFALL

**Table E-1**
**Preliminary Opinion of Probable Construction Costs for Parallel Land Outfall**

| Unit Process | Unit | Estimated Quantity | Unit Cost | Probable Construction Cost, $ |
|---|---|---|---|---|
| Mobilization | LS | 1 | 60,000 | 60,000 |
| 36" Concrete Outfall Pipeline | LF | 14,400 | 276 | 3,974,400 |
| Bore and Jack at San Joaquin River – Pipe and Installation | LF | 200 | 480 | 96,000 |
| Bore and Jack Receiving Pit | EA | 2 | 30,000 | 60,000 |
| Bore and Jack Pit Dewatering / Coffer Dam | EA | 2 | 6,000 | 12,000 |
| Blow-off Valve | EA | 2 | 600 | 1,200 |
| Air Release Valve | EA | 3 | 12,000 | 36,000 |
| 36" Flap Gate | EA | 1 | 15,000 | 15,000 |
| 36" Sluice Gate | EA | 1 | 30,000 | 30,000 |
| River Outfall Structure, concrete | CY | 3 | 840 | 2,520 |
| River Outfall Structure, electrical | LS | 1 | 12,000 | 12,000 |
| Miscellaneous | LS | 1 | 12,000 | 12,000 |
| Grand Total | | | | 4,300,000 |