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Process Definition Report

San Francisco International Airport Industrial Wastewater Treatment Plant Upgrade (Technical Memorandum 5)

Prepared for

San Francisco International Airport

September 2012

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Abbreviations and Acronyms

µm	micrometer(s)
BOD	biochemical oxygen demand
CCB	chlorine contact basin
CDPH	California Department of Public Health
CIP	clean-in-place
CT	concentration times contact time
DAF	dissolved air flotation
FRP	fiberglass reinforced pipe
ft^2	square feet
gal	gallon(s)
gph	gallon(s) per hour
gpm	gallon(s) per minute
HMI	human-machine interface
hp	horsepower
IWTP	Industrial Wastewater Treatment Plant
lb	pound(s)
LCP	Local Control Panel
LOR	Local Off-Remote
MCC	motor control center
MF/UF	micro filtration or ultra filtration
mg	milligram(s)
mgd	million gallon(s) per day
mm	millimeter
mg/L	milligram(s) per liter
MLTP	Mel Leong Treatment Plant
NTU	nephelometric turbidity units
O&M	operations and maintenance
PLC	programmable logic controller

psi	pound(s) per square inch
psig	pound(s) per square inch gauge
SCADA	supervisory control and data acquisition
SBRs	Sequencing Batch Reactors
SFIA	San Francisco International Airport
SWD	side water depth
SWTP	Sanitary Wastewater Treatment Plant
TBD	to be done
TDH	total dynamic head
TM	technical memorandum
VFD	variable frequency drive

1.0 Preliminary Process Description

1.1 Introduction

The City and County of San Francisco owns and operates the Mel Leong Treatment Plant (MLTP) through the San Francisco International Airport (SFIA). The MLTP includes an Industrial Wastewater Treatment Plant (IWTP) and a Sanitary Wastewater Treatment Plant (SWTP). The IWTP treats industrial wastewater as well as first-flush, stormwater runoff from airport facilities, and the SWTP treats the sanitary sewer from the airport facilities. The effluent from both facilities (IWTP and SWTP) is routed to either the effluent pump station or the water reclamation pump station. The combined effluent is pumped to the North Bayside System Unit outfall, where the treated water is combined with effluent from South San Francisco, Millbrae, and Burlingame. Dechlorination takes place in the shared outfall before the effluent is discharged into the Bay. The effluent from the water reclamation pump station is used within SFIA for irrigation purposes under a restricted use permit. The IWTP sludge is conveyed directly to onsite sludge drying beds for dewatering and drying. The SWTP biosolids are first sent to a belt-filter press and then to the drying beds.

Technical Memorandum (TM) 4 entitled "San Francisco International Airport Treatment Technology Selection" identified the following unit treatment processes for IWTP and tertiary treatment plant upgrades:

IWTP Unit Treatment Processes:

- Dissolved air flotation with flocculation
- Trickling filter and secondary clarification

Tertiary Treatment Unit Treatment Processes:

- Membrane filtration
- Chlorine contact basin
- Recycled water storage (a consideration will also given to provide recycled water storage tank with integrated chlorine contact basin)

The objective of this report is to define the project elements and unit treatment processes in sufficient detail. This information will be used in the Facility Design Definition phase. It also provides preliminary cost estimates for each new unit and a preliminary construction schedule. TM 5 is based on information developed in previous TMs 2 through 4, which are provided in Appendix E for reference.

1.2 Primary and Secondary Treatment Process Change Details

The following sections describe the existing IWTP treatment units and processes as well as the proposed changes.

1.2.1 Equalization Basin

The existing equalization tank is a 60-foot diameter, 30-foot tall steel tank with a total volume of 560,300 gallons. However, the wastewater flows by gravity to the rapid mixed tank such that the equalization tank can only be emptied to an elevation equal to the existing rapid mix tank #1 liquid level plus the gravity flow head loss between the two tanks. The effective storage volume is limited to 200,000 gallons. Additional equalization storage capacity is needed and will be provided as noted below.

The existing tank will be retained and refurbished. The tank is old and has operational problems such as a broken mixer, etc. The construction contractor will determine the nature and extent of refurbishment required.

In order to increase the equalization capacity of the existing equalization tank by a factor of about 2, a new pump station will be installed to pump the wastewater to the new rapid mix tanks, allowing the full volume of each equalization tank to be used if necessary. Utilization of the full volume of the equalization tank will also minimize the buildup of sludge at the bottom of the tank. Currently, the tank can only empty about half of the volume, thereby allowing sludge to build up in the tank with no easy means to remove it. A bypass valve will be installed in parallel to the new pumps station to maintain the possibility of gravity flow to the dissolved air flotation (DAF) units. An emergency bypass will also be installed to send the equalization tank effluent to any of the Sequencing Batch Reactors (SBRs), one of which will be empty at any given time to receive wastewater in an emergency.

The existing trickling filter will be used for equalization purposes while the existing equalization tank is being refurbished to allow IWTP operation during construction (see discussion below under secondary treatment).

1.2.2 Preliminary Treatment

The existing preliminary treatment facilities consist of a rapid mix tank and a flocculent tank with detention times of 5 and 10 minutes, respectively. The existing rapid mix tank is used to mix the coagulant agent and the polymer into the wastewater, with an option to also add caustic or acid. Both tanks are located in the IWTP building.

In the new IWTP, rapid mixing and flocculation steps will be integrated into each of three new DAF units as described below.

1.2.3 Primary Treatment

The existing primary treatment facilities include two DAF units. The existing DAF units do not have bottom sludge collectors to collect heavy material that settles out, requiring periodic shutdown and cleanout of the DAF units.

The new design will include three new 50 percent capacity DAF units. The proposed DAF units each include an integrated rapid mixing zone and flocculation zone and are expected to produce a thickened sludge of 2 to 4 percent. The new DAF units will be canopy covered in the new IWTP building.

DAFs can be either the circular type with a rake mechanism for sludge solids removal or the rectangular type with submerged sludge scrapers or screw conveyors for sludge solids removal. The circular design has the advantage of using well-proven, robust rake

mechanisms that operate very reliably; however, the circular design occupies a somewhat larger footprint. The rectangular design typically includes a bottom scrapper, a "V" bottom and a submerged screw conveyor, and/or automatic valves to remove sludge solids. The rectangular design occupies less space. Quotes for both types of DAF units are provided in Appendix B. SFIA staff have made the decision to go with rectangular DAF units. (Note: Type of sludge removal device is yet to be finalized.)

1.2.4 Secondary Treatment

The existing secondary treatment includes a DAF effluent pump station, rapid mix tank #2, a trickling filter, and two clarifiers. The existing rapid mix tank is used for pH control (if needed—normally bypassed) with the possibility of adding sodium hydroxide of sulfuric acid into the wastewater. The tank detention time is 3.4 minutes.

Under current operations, the wastewater is pumped from the rapid mix tank #2 to one 100 percent capacity trickling filter. The existing trickling filter is a 75-foot diameter, 13.5-foot tall steel tank filled with plastic medium. Wastewater is applied at a controlled rate with a rotary distributor at the tank top. The existing recycle flow ranges from 1.2 million gallon(s) per day (mgd) to 2.4 mgd. Two 100 percent capacity, 32-foot diameter, 15-foot tall steel tank clarifiers are used for removing total suspended solids (TSS). The sludge is currently sent to Pump Station F, from which the sludge flows into the equalization tank and subsequently to the DAF units (as floating scum or bottoms sludge) and ultimately discharges into the drying beds. The clarified effluent currently flows by gravity to a small industrial wastewater chlorine contactor and subsequently by gravity flow to the existing combined effluent pumping station.

For the new facility, one new rapid mix tank #2 will be installed (for pH control if needed—normally bypassed). It will be a 4,100-gallon steel tank with the same detention time as the existing tank at design flow.

A new 100 percent capacity trickling filter will be installed. The trickling filter tank will be constructed of steel and the size will be about 76-foot diameter by 15-foot tall, about the same as the existing trickling filter. The recirculation pumps will be set up the same way as the existing scheme, with a design flow of 1.6 mgd to insure sufficient loading. Recirculation of trickling filter effluent from the tank bottom to the top improves filter efficiency and reduces odor potential. A rotary distribution arm on top of the filter will allow a good flow distribution.

Two new 100 percent capacity concrete clarifiers will be built with a size similar to the existing ones. The clarified effluent will be pumped directly to the new tertiary treatment process. The clarifier sludge will be recycled directly to the DAF units instead of to the equalization tank to minimize solid settling and sludge buildup in the equalization tank. As an alternative scenario, capability to send the clarifier sludge to the waste sludge pump station for direct discharge to the sludge drying beds will be provided.

After the new trickling filter is installed, the existing trickling filter will be utilized as a temporary equalization tank while the existing equalization tank is being refurbished. After all construction is completed, the existing trickling filter will be abandoned in place or used for other purposes yet to be determined.

1.2.5 Disinfection System

The clarified effluent is currently sent to an existing small chlorine contact basin at the IWTP. The chlorination facilities consist of a chlorine contact basin and sodium hypochorite storage and metering system. The basin is a 15-foot by 50-foot concrete structure, placed 3 feet below grade level, with a depth of 6 feet. The chlorine contact basin provides 20 minutes of contact time.

In the new IWTP, the trickling filter effluent will be sent directly to tertiary treatment through a new pump station or to the existing combined chlorine contact basin. The disinfection process will not be part of the secondary treatment units.

1.2.6 Sludge Handling Facility

Under current operations, sludge from the DAF process flows by gravity to the sludge wet well. From there, it is pumped to the sludge drying beds for dewatering. The clarifier sludge is pumped to the equalization tank and recirculated through the plant.

The same configuration for sludge pumping will be retained in the new IWTP. In the new ITWP, the clarifiers sludge will be sent directly to the DAF units. Sludge will be produced on the DAF units' tops and bottoms. It will be pumped similarly to the existing drying beds through a waste sludge pit. The volume of the new waste sludge pit will be twice as big as the existing one, as requested by facility staff.

It is recommended that the clarifier sludge be recycled directly to the DAF units instead of to the equalization tank to minimize solid settling and sludge buildup in the equalization.

1.2.7 Chemical Facilities

The existing system includes storage tanks for caustic, alum, and polymer storage and mixing. The system also includes feeders, mixers, metering pumps, flow meters, day tanks, and associated controls. The chemical facilities will be installed in a separate enclosed room attached to the new DAF canopy building. A nominal 45-day supply of chemicals will be provided. Although not required for routine usage, a small acid metering system (for example, carboy size) will be included for emergency pH control.

1.2.8 Clarifier Effluent Pump Station

The existing clarified effluent pump station will be replaced or refurbished to pump clarifier effluent to the new tertiary treatment system or to the combined chlorine contactor. The pump station will have a design flow of 1.7 mgd.

1.3 Tertiary Treatment Process Change Details

The proposed tertiary treatment plant will produce up to 1.0-mgd disinfected tertiary recycled water for unrestricted nonpotable uses including landscape irrigation and urban reuse. The tertiary treatment plant will be designed to handle both ITWP and SWTP secondary effluent. However, as noted in TM 3 and TM 4, the SWTP secondary effluent will be used as the primary source for the tertiary treatment. Furthermore, the quality of the SWTP secondary effluent is such that it controls the design of the tertiary treatment plant design. The SWTP secondary effluent has an average turbidity of 11 nephelometric turbidity units (NTU) with periodic excursion exceeding 20 NTU, which is more challenging to treat

and meet Title 22 recycled water criteria compared to treating IWTP secondary effluent. The tertiary treatment plant will include membrane filtration, chlorine disinfection, and recycled water storage.

1.3.1 Membrane Filtration

Membrane filtration, micro filtration or ultra filtration (MF/UF), is a pressure-driven separation process that typically employs hollow-fiber membranes to block the passage of solids (i.e., turbidity, suspended solids) and pathogenic microorganisms, including bacteria and protozoa. While MF pore sizes range from about 0.1 to 0.2 micron (nominally 0.1 micron), UF pore sizes range from 0.01 to 0.05 micron (nominally 0.01 micron). Despite the pore-size differences, both systems produce virtually solids-free effluent with a turbidity typically less than 0.1 NTU without chemical addition for particle coagulation.

MF/UF can be configured as both pressurized (encased) or submerged systems. MF/UF systems are also classified based on flow pathway (outside-in vs. inside-out). For high solids feed water applications (i.e., reuse application), outside-in MF/UF systems generally perform better than inside-out configurations. In both types of systems, membrane fibers are bundled in groups of several thousand and potted in a resin on both ends to form a module, with tens to hundreds of modules coupled together to form a system. With pressurized type, the modules are housed in a pressure vessel, or the vessel is integral to the module. Feed water is pressurized and applied to the feed side of the membranes in the module. Typical operating pressures range from 3 to 40 pounds per square inch (psi), depending on membrane technology and specific product operating conditions.

MF/UF systems are designed to filter small suspended solids and particles. Larger-size suspended solids, if allowed to enter the fiber bundle, can cause fiber damage (including breakage), and accumulate, leading to a buildup of solids. As a result, MF/UF systems employ self-cleaning strainers with a screen size of 500 microns or less to prevent entry of larger particulates into the membrane modules. As constituents accumulate on the membranes, modules need to be backwashed or cleaned chemically to prevent membrane fouling. The backwash flows will be returned to the head of the SWTP for further processing. Chemical cleaning waste will be neutralized and returned back to the head of the SWTP.

For small applications (<1.5 mgd), packaged membrane systems are more cost effective than custom design systems. In addition, pressurized membrane systems require less footprint and are typically more cost effective than submerged membrane systems for small applications (<5 to 10 mgd). Therefore, in this project, packaged and pressurized membrane systems will be considered. Because membranes are proprietary, the design filtration (flux), backwash, and chemical cleaning regimes may vary among the suppliers. The design criteria in this TM is based on a packaged MF system (AP-6) provided by Pall Microfiltration System. It should be noted that other membrane systems supplied by Siemens, GE Water and Process, DOW, Toray, and California Department of Public Health (CDPH)-approved other membranes are also qualified and can be used in this project.

Two membrane trains (each has capability to produce 0.5 mgd) will be provided. The treated (filtered) flow will enter the chlorine contact basin influent channel. Since the filtrate from the pressurized MF/UF systems will have a residual pressure of minimum 10 to

15 pounds per square inch (psi), which is enough to convey flow to the chlorine contact basin without additional pumping.

1.3.2 Chlorine Disinfection

The filtered wastewater will be disinfected in a dedicated chlorine contact basin (CCB). The new CCB for the tertiary treatment plant will be sized using tertiary peaking factor of 1.5 to satisfy peak recycled water demand. Chlorine in the form of gaseous chlorine or liquid sodium hypochlorite is injected into the wastewater. Chlorine disinfection should provide a minimum concentration times contact time (CT) requirement of 450 milligrams (mg) per minute per liter with a modal contact time of no less than 90 minutes enforced by the CDPH. A tracer testing is required to determine modal contact time for the CCB. Current design practices generally use 120-minute hydraulic detention time in lieu of 90-minute modal contact time for well-baffled CCBs, which form a plug flow to provide adequate contact between disinfectant and wastewater pathogen.

The chlorine demand for the tertiary plant will be calculated by a programmable logic controller (PLC) based on the chlorine dose setpoint and filtrate flow, with the chlorine residual analyzers (at the end of the new CCBs) providing the trimming signal to control the chlorine dose.

The inlet to the new CCBs will consist of sluice gates to minimize off-gassing. A motor-operated sluice gate connected to a pipe will be provided at the end of the last pass of the CCB to allow out-of-compliance water to be conveyed to the plant outfall.

A dedicated chlorine feed and storage system will be provided for tertiary plant disinfection system. Liquid sodium hypochlorite (12.5 percent) will be used for disinfection. The chlorine feed and storage system will consist of one bulk tank, one day tank, and chemical transfer and metering pumps (each will arrange as one duty plus one standby).

1.3.3 Recycled Water Storage

For most recycled schemes (i.e., landscape and agricultural irrigation), the recycled water demand usually occurs only during a portion of the day, thereby requiring water storage to satisfy the demand. The SFIA staff indicated that the recycled water can be distributed to the users throughout the day, which reduces the need for recycled water storage. With the lack of detailed recycled water, demand data, and per SFIA staff input, a recycled water storage volume of 200,000 gallons was assumed.

For recycled water storage, there are two viable options:

- A concrete tank adjacent to the chlorine contact basin tank effluent
- A storage tank with integrated CCB

In the second option, a 325,000-gallon storage tank will be constructed and baffled to simulate the CCB flow pattern to enhance contact between chemical and microorganisms to ensure adequate inactivation of pathogens. The baffle walls will be extended almost to the tank height and a level sensor will be provided to maintain the CCB volume. A minimum volume of 125,000 gallons will be maintained and dedicated for the CCB, while the remaining volume of 200,000 gallons will serve as storage.

One main advantage of the second option over the first one is that it combines both CCB and recycled water in a single tank that reduces footprint requirement. On the other hand, the second option requires pumping after CCB, which contributes to both capital and operations and maintenance (O&M) cost of the facility. Comparison of capital costs (include contractor markups and 25 percent project contingency) revealed that Option 1 ($1,465,000) is slightly better option than Option 2 ($1,549,000) offering about $84,000 savings in capital investment. Option 1 has slightly lower O&M cost than Option 1 due to exclusion of intermediate pumping. Despite the cost disadvantage, the footprint available in the site may favor Option 2 over Option 1. Therefore, footprint and facility requirements will be developed for the two options considered.

1.3.4 Recycled Water Pump Station

A new recycled water pump station will convey recycled water to SFIA's users. The variable frequency drive (VFD)-driven pumps will operate to maintain a desired pressure setpoint in the recycled water distribution system. Because the recycled water will mainly be used within the SFIA boundary and its close proximity, the hydraulic grade lines for the recycled water users may be very similar, which simplifies pump station and distribution system design. A TDH of 150 feet was assumed for sizing the recycled water pump station (to be verified in forthcoming task). Two pumps (one duty and one standby), along with a common jockey pump, are selected for the design. The recirculation piping system consists of a branch on the main discharge header with a pressure reducing valve and manual isolation valve that will return the recycled water back to the recycled water storage tank. The recirculation piping system will be used when the actual recycled water demands are below the minimum operating flow of the jockey pump.

1.4 IWTP Upgraded Overall Process

1.4.1 Preliminary Mass Balance

Process schematics with flow, TSS, and biochemical oxygen demand (BOD) information for the primary, secondary, and tertiary treatment are provided in Appendix A.

The SWTP secondary effluent will be used as the primary source for the tertiary treatment. Considering SFIA objectives to reduce NPDES discharges and maximize recycled water production in the near future, the tertiary treatment plant will be designed to treat both IWTP and SWTP secondary effluent. TM 4, San Francisco International Airport Treatment Technology Selection, identified that membrane filtration is the most suitable and cost-effective technology to meet turbidity criteria of Title 22 recycled water regulations. Because membrane filtration is an absolute barrier to solids, chemical pretreatment for solids removal is not required to meet Title 22 turbidity requirements. Membranes require frequent backwashes and periodic chemical cleanings during which no water is produced. To satisfy redundancy criteria of Title 22 recycled water regulations, two identical membrane trains will be provided that will satisfy average recycled water demand as one train out of service. While the chlorine feed pumps are configured as duty and standby arrangements, no redundant train for CCB will be provided, because CCB itself has no mechanical or electrical component that requires routine maintenance. Appendix A shows the process schematics and flows and solids mass balances under design flows (1.0 mgd recycled water produced) for the two train options (one with separate CCB and recycled

water storage and the other with combined CCB and water storage). The backwash streams from strainers and MF will be returned back to the SWTP headworks. The chemical cleaning wastes from MF will be neutralized before being returned to the SWTP headworks.

1.4.2 Primary and Secondary Treatment Design Criteria

Preliminary design criteria for the major equipment associated with the IWTP upgrade is provided in Table 1-1. Appendix B includes equipment information from vendor quotes.

TABLE 1-1
Primary and Secondary Treatment Design Criteria

Design Criteria	Value
Equalization Tank Pump Station	
Number of Feed Pumps	3 (2 duty+1 standby)
Pump TDH, each, feet	35
Capacity, each, mgd	0.85
Brake Motor, HP, each	7.0
Pump HP, each	10.0
Alum Pump Station	
Design Flow, mgd	1.7
Design Dosage, mg/L	50 (30 - 70)
Design Daily Usage, lb/day	709
Effective (Active) Density, lb/gal	5.42
Design Daily Usage, gal/day	131
Number of Feed Pumps	2 (1 duty+1 standby)
Capacity of Pump, gph, each	10
Type	Hydraulically actuated flat diaphragm
Pump Motor	1/2 hp
Alum Storage	
Number of tanks	1
Tank volume, each, gal	5,000
Tank description	Fiberglass
Minimum Storage Provided at Design Flows and Dose, days	38
Also Includes: feeder, mixer, metering pump, flowmeter, day tank, and control	
Polymer Feed and Storage System	
Polymer Pump Station	
Design Flow, mgd	1.7
Design Dosage, mg/L	60 (40 - 80)
Design Daily Usage, lb/day	851
Effective (Active) Density, lb/gal	9.17
Design Daily Usage, gal/day	93
Number of Feed Pumps	2 (1 duty+1 standby)
Capacity of Pump, gph, each	5
Type	Hydraulically actuated flat diaphragm
Pump Motor	1/2 hp
Polymer Storage	
Number of tanks	2
Tank volume, each, gal	133
Tank description	Fiberglass for polymer mixing and aging
Also Includes: feeder, mixer, metering pump, flowmeter, day tank, and control	
Rapid Mix Tank No.2	
Number of tanks	1
HRT, min	3.4
Tank volume, each, gal	4,100

TABLE 1-1
Primary and Secondary Treatment Design Criteria

Design Criteria	Value
DAF Units	
Mixing Integrated Unit	
Flocculation Integrated Unit	
DAF Unit (rectangular, V Bottom with screw conveyor-pending)	3 @ 50% Capacity each
DAF Unit Dimensions, feet wide x feet long	8 x 58 (Westech) 12 x 22 (World Water Works)
Recycle pumps	Included
Scum/sludge Pump Station	
Number of Feed Pumps	3 (2 duty+1 standby)
Pump TDH, each, ft	20
Capacity, each, mgd	0.0035
Brake Motor, HP, each	0.016
Pump HP, each	5.0
Primary to Secondary Pump Station	
Number of Feed Pumps	3 (2 duty+1 standby)
Pump TDH, each, ft	35
Capacity, each, mgd	0.85
Brake Motor, HP, each	7.0
Pump HP, each	10.0
Acid Feed System (Sulfuric Acid)	
Acid Metering Station	
(For emergency use only)	
Number of pumps	1
Type	Metering Pump
Number of tanks (carboys)	1
Tank volume, each, gal	100 to 250
Tank description (provided by chemical supplier)	Carboy
Caustic Feed and Storage System	
Caustic Pump Station	
Design Flow, mgd	1.7
Design Dosage, mg/L	3
Design Daily Usage, lb/day	43
Effective (Active) Density, lb/gal	6.42
Design Daily Usage, gal/day	7
Number of Feed Pumps	2 (1 duty+1 standby)
Capacity of Pump, gph, each	5
Type	Hydraulically actuated flat diaphragm
Pump Motor	1/2 hp
Caustic Storage	
Number of tanks	1
Tank volume, each, gal	1,600
Tank description	FRP or HDPE
Minimum Storage Provided at Design Flows and Dose, day	241
Also Includes: feeder, mixer, metering pump, flowmeter, day tank, and control	
Trickling Filter	
Tank	
Number of Units	1 steel tank
Tank dimensions, each, feet diameter x ft height	76 x 15
Packing Media	
Media volume, $feet^3$	46,720

TABLE 1-1
Primary and Secondary Treatment Design Criteria

Design Criteria	Value
TF Flow Recirculation Pumps	
Number of Feed Pumps	3 pumps (2 duty+1 standby)
Feed Pump TDH, each, feet	20
Design flow (50%), each, mgd	0.82
Pump HP,, each	25
Distribution Arm	
Secondary Clarifier	
Number of Units	2
Tank dimensions, each, feet diameter x feet height	32 x 15
Sludge Recirculation Pump Station	
Number of Feed Pumps	3 (2 duty+1 standby)
Pump TDH, each, feet	35
Capacity, each, mgd	0.0035
Brake Motor, HP, each	0.029
Pump HP, each	5.0
Pump Station to Tertiary Treatment	
Number of Feed Pumps	1 duty + 1 standby and a small Jockey pump
Pump Type	Vertical Turbines
Pump TDH, each, feet	150
Capacity, each, mgd	1.7
Brake Motor, HP, each	59.6
Pump HP, each	70.0
Drive	VFD

1.4.3 Tertiary Treatment Design Criteria

Tables 1-2 and 1-3 present tertiary treatment design criteria for the two options presented in Appendix A. The membrane filtration design criteria in Table 1-2 are based on Pall Corporation AP 60 packaged pressurized MF system. Other CDPH packaged membrane filtration systems (MF or UF) utilizing CDPH-approved membranes provided by GE Water Technologies, Siemens, DOW, Toray, and others may also be considered for facility design.

TABLE 1-2
Tertiary Treatment Design Criteria for MF/UF,CCB, and Separate Recycled Water Storage

Design Criteria	Value
Membrane Pretreatment-Self-Cleaning Strainer	
Design Feed Flow, mgd	1.06
Clean strainer pressure drop at the design flow, psig	1
Projected backwash waste (% of the feed)	0.5
Screen Size, µm	300
Expected Head Loss, psi	<2 psi
Type	Automatic self cleaning
Number of Strainers	2 (1duty+1 standby)
Motor Size, hp	0.25
Membrane System[1]	
Design feed flow, mgd	1.05
Design filtrate flow, mgd	1.00

TABLE 1-2
Tertiary Treatment Design Criteria for MF/UF,CCB, and Separate Recycled Water Storage

Design Criteria	Value
Membrane Modules	
Type	AP-6 Package
Nominal Pore Size, micron	0.1
Membrane Material	PVDF
Membrane Area per Module, ft^2	538
Number of membrane trains	2
Number of Modules Per Train	60
Membrane Module Area per Train, ft^2	32,280
Flux with one train out of service at Design Flow, gpd/ft^2	31
Recovery, %	95
Maximum TMP, psi	40
Membrane Feed Pumps	
Number of Feed Pumps	One per Train (1 shelf spare)
Feed Pump TDH, each, feet	110
Horsepower, each, HP	30
Drive	VFD
Reverse Flow Pumps	
Number of Feed Pumps	One per Train (1 shelf spare)
Feed Pump TDH, each, feet	69
Horsepower, each, HP	20
Drive	VFD
Reverse Flow Tanks	
Number of Tanks	One per Train (1 shelf spare)
Material	HDPE
Volume, gal	1,100
Compressed Air System	
Number of Compressors	2 (1 Duty+1 standby)
Type	Rotary Screw
Number of Receivers	1
Type	Vertical
Pressure rating, psi	200 (ASME Coded)
Drain Valve, inches	0.5
Drain Valve Type	Ball Valve
CIP Skid (One Common Skid)	
Number of CIP Tank	1
Volume of Tank, gal	2,500
Material of Construction	HDPE
CIP Recirculation Pumps	
Number of Recirculation Pumps per Train	One per Train (1 shelf spare)
Capacity, each, gpm	120
Motor, hp	3
Drive	VFD
Neutralization Skid (One Common Skid)	
Number of Neutralization Tank	1
Volume of Tank, gal	8,750
Material of Construction	HDPE
Diameter, feet	12
Tank SWD, feet	10.4
Tank Shell Height, feet	12.4
Volume, gal	8,800

TABLE 1-2
Tertiary Treatment Design Criteria for MF/UF,CCB, and Separate Recycled Water Storage

Design Criteria	Value
CIP Chemical Transfer Pumps	
Number of Chemical Feed Pumps	3 (1 shelf spare)
Capacity, gpm, each	3
Motor, hp	0.5
Chlorine Disinfection	
Chlorine Feed System	
Design Chlorine Dose, mg/L	10
Design Chlorine Dose for Chlorine Residual	2
Chlorine Strength, %	12.5
Design Flow, mgd	1.00
Design Daily Sodium Hypochlorite Usage for Disinfection, lb/day	83
Design Daily Sodium Hypochlorite Usage for Disinfection, gal/day	66.2
Number of Tanks	1
Type	FRP
Diameter, feet	10
Tank SWD, feet	6
Tank Shell Height, feet	8
Volume, gal	3,520
Storage Provided at Average Flow, days	53
Chlorine Metering Pumps	
Number of Metering Pumps	2 (1 duty+1 standby)
Capacity, each, gph	10.0
hp, each	0.25
Chlorine Contact Basin	
Design Flow, mgd	1.0
Peak Hourly Flow at Tertiary peaking Factor of 1.5, mgd	1.5
Minimum Detention Time, minutes	120
Minimum CT, mg/L*minutes	450
Basin Volume, gal	125,000
Number of Basins	1
Number of Passes per Basin	3
SWD, feet	9
Length, feet	75
Width of Each Pass, feet	9
Total Volume Provided, gal	136,400
Chlorine Contact Basin Influent Channel	
Channel width, feet	10
Channel Length, feet	31
Channel Depth, feet	9
Recycled Water Storage Tank	
Channel width, feet	31
Channel Length, feet	60
Channel Depth, feet	15
Desired Volume, gal	200,000
Volume Provided, gal	208,700
Recycled Water Pump Station	
Design Flow, mgd	1.5
Design TDH, feet	150 (to be verified during facility design)
Number of Pumps	2 (1 duty+1 standby) plus one small jockey pump
Capacity, each, gpm	1,100
Motor, each, hp	60

[1] Membrane system design is based on Pall Corporation AP 60 Packaged MF system. Other packaged membrane filtration systems with CDPH-approved membranes are qualified and can be used during facility design.

TABLE 1-3
Tertiary Treatment Design Criteria for MF/UF, CCB, and Water Storage with Integrated CCB

Design Criteria	Value
Membrane Pretreatment-Self-Cleaning Strainer	
Design Feed Flow, mgd	1.06
Clean strainer pressure drop at the design flow, psig	1
Projected backwash waste (% of the feed)	0.5
Screen Size, µm	300
Expected Head Loss, psi	<2 psi
Type	Automatic self cleaning
Number of Strainers	2 (1duty+1 standby)
Motor Size, hp	0.25
Membrane System[1]	
Design feed flow, mgd	1.05
Design filtrate flow, mgd	1.00
Membrane Modules	
Type	AP-6 Package
Nominal Pore Size, micron	0.1
Membrane Material	PVDF
Membrane Area per Module, ft^2	538
Number of membrane trains	2
Number of Modules Per Train	60
Membrane Module Area per Train, ft^2	32,280
Flux with one train out of service at Design Flow, gpd/ft^2	31
Recovery, %	95
Maximum TMP, psi	40
Membrane Feed Pumps	
Number of Feed Pumps	One per Train (1 shelf spare)
Feed Pump TDH, each, feet	110
Motor, each, hp	30
Drive	VFD
Reverse Flow Pumps	
Number of Feed Pumps	One per Train (1 shelf spare)
Feed Pump TDH, each, ft	69
Motor, each, hp	20
Drive	VFD
Reverse Flow Tanks	
Number of Tanks	One per Train (1 shelf spare)
Material	HDPE
Volume, gal	1,100
Compressed Air System	
Number of Compressors	2 (1 Duty+1 standby)
Type	Rotary Screw
Number of Receivers	1
Type	Vertical
Pressure rating, psi	200 (ASME Coded)
Drain Valve, inches	0.5
Drain Valve Type	Ball Valve
CIP Skid (One Common Skid)	
Number of CIP Tank	1
Volume of Tank, gal	2,500
Material of Construction	HDPE

TABLE 1-3
Tertiary Treatment Design Criteria for MF/UF, CCB, and Water Storage with Integrated CCB

Design Criteria	Value
CIP Recirculation Pumps	
Number of Recirculation Pumps per Train	One per Train (1 shelf spare)
Capacity, each, gpm	120
Motor, each, hp	3
Drive	VFD
Neutralization Skid (One Common Skid)	
Number of Neutralization Tank	1
Volume of Tank, gal	8,750
Material of Construction	HDPE
Diameter, feet	12
Tank SWD, feet	10.4
Tank Shell Height, feet	12.4
Volume, gal	8,800
CIP Chemical Transfer Pumps	
Number of Chemical Feed Pumps	3 (1 shelf spare)
Capacity, each, gpm	3
Motor, each, hp	0.5
Intermediate Pump Station	
Design Flow, mgd	1.00
Design TDH, feet	40
Number of Pumps	2 (1 duty+1 standby)
Capacity, each, gpm	750
Motor, each, hp	15
Chlorine Feed System	
Design Chlorine Dose, mg/L	10
Design Chlorine Dose for Chlorine Residual	2
Chlorine Strength, %	12.5
Design Flow, mgd	1.00
Design Daily Sodium Hypochlorite Usage for Disinfection, lb/day	83
Design Daily Sodium Hypochlorite Usage for Disinfection, gal/day	66.2
Number of Tanks	1
Type	FRP
Diameter, feet	10
Tank SWD, feet	6
Tank Shell Height, feet	8
Volume, gal	3,200
Storage Provided at Average Flow, days	53
Chlorine Metering Pumps	
Number of Metering Pumps	2 (1 duty+1 standby)
Capacity, each, gph	10.0
Motor, each, hp	0.25
Chlorine Contact Basin	
Recycled Water Storage with Integrated Chlorine Contact Basin	
Design Flow, mgd	1.0
Peak Hourly Flow at Tertiary peaking Factor of 1.5, mgd	1.5
Minimum Detention Time, minutes	120
Minimum CT, mg/L*minutes	450
Minimum Basin Volume Required for CCB, gal	125,000
Storage Volume, gal	200,000
Total Tank Volume, gal	325,000
Number of Tanks	1
Diameter, feet	50
Tank SWD, feet	23

TABLE 1-3
Tertiary Treatment Design Criteria for MF/UF, CCB, and Water Storage with Integrated CCB

Design Criteria	Value
Tank Shell Height, feet	25.5
Total Volume Provided, gal	337,700
Recycled Water Pump Station	
Design Flow, mgd	1.5
Design TDH, ft	150 (to be verified during facility design)
Number of Pumps	2 (1 duty+1 standby) plus one small jockey pump
Capacity, each, gpm	1,100
Motor, each, hp	60

[1] Membrane system design is based on Pall Corporation AP 60 Packaged MF system. Other packaged membrane filtration systems with CDPH-approved membranes are qualified and can be used during facility design.

1.5 Power Supply and Distribution System

This section will be completed upon confirmation of the above.

1.5.1 Preliminary Load Analysis

1.5.2 Primary Power Supply

1.5.3 Standby Power System

1.6 Site Layout and Hydraulic Profile

A preliminary site layout is provided in Appendix C. The wastewater is expected to either flow by gravity or be moved by pumps. The equalization tank effluent will be pumped to the DAF units. The DAFs effluent will flow by gravity to Rapid Mix Tank #2 and then will be pumped to the trickling filter. The DAFs scum and sludge will be pumped to the drying beds. The secondary clarifiers will be partially underground, so that the trickling filter effluent will flow to them by gravity. Sludge from the secondary clarifiers will be pumped and recirculated to the DAF units. A pumps station will send the secondary clarifier effluent to the tertiary treatment location or to the combined chlorine contactor.

1.7 Preliminary Process Control Narratives

1.7.1 Equalization Tank

After refurbishment, the existing equalization tank and associated pump station will be controlled based on tank level control. The associated pumps will have VFDs.

1.7.2 DAF Units

The following description corresponds to a standard control system for these units.

In the flocculation mixing chamber, the mixer gently mixes inlet stream to promote particle growth. The flocculator drive unit is mounted on the tank side wall. The flocculator drive unit consists of a VFD-rated motor coupled to a constant speed reducer. The flocculated stream is sent to the floatation chamber. These flocs are floated to the surface where they are collected by the skimmer system and moved to the float chamber for removal. Heavier solid particles that settle to the floor of the tank are removed by using drain valves. Clarified

effluent flows under a baffle and over an adjustable weir at the discharge end of the DAF clarifier.

Part of the clear effluent is withdrawn from the effluent collection box and directed through recycle pumps into the saturation tank. Within the saturation tank, air is dissolved into the liquid under pressure. The liquid level within the tank is controlled by the amount of air and the air release valve. The air saturated liquid then exits the saturation tank and is directed toward the back pressure control valve. This valve is designed to create very fine air bubbles as the saturated air is suddenly released from the liquid. The valve is placed such that the air bubbles are released near the inlet of the sludge.

The pressurization system includes a recycle pump that sends part of the effluent into a saturation tank. A torque box automatically stops the drive in the event that the torque limit is exceeded. The air is dissolved into the water, and the pressurized mixture is then directed into the back pressure pipe inlet. Pressure gauges measure pressure in the primary recycle line after the primary recycle pump saturation tank and in the piping before entering a back-pressure control valve.

An air flow control panel regulates and measures air flow and pressure to the saturation tank. The control panel includes a pressure regulator, a solenoid valve connected to the recycle pump and a rotameter.

The saturation tank is equipped with a level controller that controls the water level in relation to air flow and pressure. A pressure relief valve is installed for safety. The sight glass visually shows the level of liquid in the saturation tank and indicates a high water alarm.

An electrical control panel monitors operation of the DAF mechanism.

1.7.3 Trickling Filter

Trickling filters are a simple biological treatment and require minimal process control instrumentation. The arm rotary distributor will be driven by hydraulic head. The recycling pumps will be equipped with VFDs, and the recycling flow should be adjusted to maintain an adequate loading/wetting. A flowmeter on the pipe discharge will be required, so that operators can either input the speed of the pumps and change it when necessary or they can input a flow setpoint, and the recirculation pump speed is controlled to achieve the flow setpoint.

1.7.4 Clarifiers

Secondary clarifiers monitoring and control takes many forms and can range from complete manual operation to complete automation of clarifier sludge collection and withdrawal and control of the clarifier's sludge blanket level. Due to the size of the plant, limited automation is recommended for this application. Removal of sludge in a manner that allows maximizing solids concentration, while minimizing the effect of sludge removal on effluent water quality, is the principal task of the controls. Typically, sludge and scum will be removed alternatively based on levels in the skimming wells.

Because the sludge generation and withdrawal rates can vary over time as a result of fluctuations of influent water quality and quantity, frequent monitoring of the concentration

of sludge removed from the clarifiers and clarifier sludge blanket will be needed to provide an indication of the overall clarifier performance.

1.7.5 MF/UF Feed Strainers

Operators can locally monitor differential pressure across each of the motorized automatic strainers. Differential pressure signals will be transmitted to the plant PLC. The backwash cycle will be triggered based on either differential pressure or a predetermined time interval. Once a backwash cycle is triggered, the motorized drain valve located on the equipment will be opened for a period of time that is factory selected. However, this time period is field adjustable. One local control panel (LCP) will be provided with the strainers. This LCP will control the backwash cycle for each strainer. The pressure differential value and the strainer ON status will be displayed in the human-machine interface (HMI).

- LOCAL EQUIPMENT MONITORING AND CONTROL
  - Operators can locally monitor the differential pressure across each strainer. The differential pressure signal will be provided to the plant PLC.
  - Operators can locally operate strainers at the common LCP provided with the strainers. ON statuses for the strainers will be provided to the plant PLC.
- PCS MONITORING AND CONTROL
- PCS MANUAL – None.
- PCS AUTOMATIC
  - The PCS will measure, scale, indicate, and trend the strainers differential pressures.
  - The PCS will indicate the ON status for the strainers.

1.7.6 MF/UF Skids

There will be a magnetic flowmeter and modulating valve (motor operated) upstream of each of the two membrane filter skids to control the flow of secondary effluent to each of the skids. The operator will be able to set the flow to be treated by each MF/UF skid at the HMI.

Each of the modulating valves supplying secondary effluent to the membrane filtration skids can be manually controlled at their respective local control stations by placing the LOCAL-OFF-REMOTE (LOR) switch in LOCAL and pushing the OPEN-STOP-CLOSE pushbuttons. By placing the LOR selector switch at the local control station to the REMOTE position, each valve position can be controlled manually at the supervisory control and data acquisition (SCADA) system by selecting MANUAL from their AUTO/MANUAL selector and activating their POSITION CONTROL or OPEN/CLOSE software buttons at the HMI.

The minimum and maximum travel can also be set manually at the HMI for each of the valves. When the LOR switch for the modulating valve is in REMOTE, the modulating valve will be controlled to maintain its corresponding secondary effluent flow set point, which will be entered by the operator at the HMI. The flowmeter corresponding to the modulating valve will control the valve to maintain the flow set point.

Each MF/UF skid will be provided with an LCP housing, a PLC, and control switches to control and monitor system performance and backwash cycles. Each MF/UF skid PLC shall receive the following input from the Plant PLC for operation of the system:

- CLOSED position for the inlet modulating valve for the MF/UF skid. If in the CLOSED position, the corresponding MF/UF skid shall STOP running.
- OPEN position of the inlet modulating valve for the MF/UF skid. If not in the CLOSED position, the corresponding cloth media filter skid shall be allowed to run.
- HIGH-HIGH level alarm from the storage tank shall STOP any operating MF/UF trains.
- LOW level signal from the storage tank shall increase the amount of flow to the operating MF/UF trains.
- MEDIUM-HIGH level signal from the storage tank shall decrease the amount of flow to the MF/UF trains.
- HIGH level signal from the storage tank shall STOP any operating MF/UF trains.
- HIGH-HIGH level signal from the storage tank shall ALARM and STOP any operating MF/UF trains.

The HMI will monitor the following signals from each of the MF/UF skids LCPs:

- MF/UF system ON
- MF/UF system FAIL
- All valve positions (OPEN/CLOSE/FAIL), backwash cycles (ON/OFF), backwash pumps (ON/OFF/FAIL), and skid tank level

The two main operating modes for the MF/UF skids are [AUTO] and [MANUAL]. A third mode is labeled [DISABLED], meaning the machine is off and neither the Auto nor the Manual mode is selected. [MANUAL] mode is only necessary for setup, troubleshooting and maintenance.

- LOCAL EQUIPMENT MONITORING AND CONTROL
  - Operators can locally monitor the MF/UF feed water turbidity analyzer provided as part of the MF/UF package system. The analyzer signal will be transmitted to the MF/UF PLC. The flow to the analyzer is set locally via a rotameter dedicated to the analyzer.
  - Operators can locally monitor the MF/UF effluent flow provided as part of the MF/UF package system.
  - Operators can locally monitor the MF/UF effluent turbidity analyzer provided as part of the MF/UF package system. The analyzer signal will be transmitted to the MF/UF PLC. The flow to the analyzer is set locally via a rotameter dedicated to the analyzer.

- PCS MONITORING AND CONTROL
  - The manufacturer of the MF/UF system supplier will provide the entire control package for their respective systems, including the required instrumentation, control and remote I/O panels, and PLC for automatic and manual operation.
  - Data is transmitted via an Ethernet link between the MF/UF system master PLC panel and the plant PLC.
  - The MF/UF effluent flow signal will be transmitted to the plant PLC. Other signals to transmit between the PCS and the MF/UF system will be determined as described in the procurement specifications for the MF/UF system and as determined by coordination with Pall Corporation, the MF/UF system supplier.
  - Total feed flow from the lift pump stations shall be transmitted from the PCS to the MF/UF system via the Ethernet link.
- PCS MANUAL - None.
- PCS AUTOMATIC
  - The PCS will measure, scale, indicate, and trend the MF/UF feed water turbidity.
  - The PCS will measure, scale, indicate, and trend the MF/UF effluent turbidity. An alarm will be provided at the HMI after a time delay when a turbidity set point of 0.5 NTU is exceeded.

1.7.7 Intermediate Pump Station (Optional – Valid for Recycled Water Storage Tank with Integrated CCB)

An intermediate pump station will lift membrane-treated water to the recycled water storage basin with integrated CCB.

Each pump will be started and stopped at the local control station by placing its LOR switch to LOCAL and by using the START/LOCKOUT/STOP pushbuttons. Each pump will operate at the speed set point, in this mode. In REMOTE AUTO, the VFD will operate at the speed set by the SCADA system to maintain the level set point in the recycled water storage tank. In REMOTE MANUAL mode, the operator can adjust the speed of each pump at the VFD control panel. In the REMOTE MANUAL mode, each of the pumps will be stopped by pushing the STOP button.

Control of each pump will be transferred to the SCADA system by placing the LOR selector switch at its local control station in REMOTE. In this mode, the pumps will operate to maintain the level set point in the recycled water storage tank. This includes starting and stopping the pumps and ramping their speed up and down.

The interface for the pump station control system includes:

- HIGH-HIGH level alarm from the existing wet well that will house the pumps shall STOP any operating MF/UF trains

The following alarms and parameters will be monitored for each of the pumps:

- Motor ON status

- REMOTE status
- VFD SPEED
- VFD FAIL alarm

In addition, the level in the recycled water storage basin will be monitored and displayed.

1.7.8 Sodium Hypochlorite Injection

A new sodium hypochlorite storage and feed system will be used for disinfecting filtered water. One duty and one standby pump will be dedicated to disinfection. The sodium hypochlorite will be injected upstream of the CCB or recycled water storage with integrated CCB. Each of the SHC feed pumps will be started and stopped at the local control station by placing its LOR switch to LOCAL and by using the START/LOCKOUT/STOP pushbuttons. Each SHC feed pump will operate at the speed set point, in this mode.

- In REMOTE AUTO mode, the VFD for the disinfection SHC feed pump will operate at the speed set by the SCADA system based on the filter effluent flow (flow pace function based on the flowmeter downstream of the MF/UF and the chlorine residual immediately upstream of the recycled water pump station (trim function). In REMOTE MANUAL mode, the operator can adjust the speed of the sodium hypochlorite feed pump at the VFD control panel. In the REMOTE MANUAL mode, the disinfection feed pump will be stopped by pushing the STOP button.

Control of each SHC feed pump will be transferred to the SCADA system by placing the LOR selector switch at its local control station in REMOTE as described above. This includes starting/stopping each sodium hypochlorite feed pump and ramping their speed up and down.

Below are the interfaces for the sodium hypochlorite feed pumps:

- Filter effluent NO flow signal (from the flowmeter downstream of the MF/UF) shall STOP the sodium hypochlorite feed pump.
- HIGH-HIGH level alarm from the recycled water storage tank shall STOP the sodium hypochlorite feed pump.

The following alarms and parameters will be monitored for each of the sodium hypochlorite feed pumps:

- Motor ON status
- REMOTE status
- VFD SPEED
- VFD FAIL alarm

In addition, the level in the chlorine contact basin effluent channel.

1.7.8.1 CCB/Recycled Water Storage Tank

The new CCB will be provided with baffles, a level transmitter, and a chlorine residual analyzer on the effluent nozzle piping. Chlorinated effluent will be collected by the recycled water storage tank, which will be adjacent to the effluent section of the CCB.

The interfaces for the CCB/Recycled Water Storage Tank are:

- LOW-LOW level signal from the level transmitter shall STOP the recycled water pumps.
- LOW level signal from the recycled water storage tank shall increase the amount of flow to the MF/UF trains.
- MEDIUM-HIGH level signal from the level transmitter shall slow down the amount of flow to the MF/UF trains.
- HIGH-level signal from the level transmitter shall STOP any operating MF/UF trains.
- HIGH-HIGH level signal from the level transmitter shall ALARM and STOP any MF/UF trains.

The following status and alarms will be monitored: level in the recycled water storage tank, LOW-LOW level and alarm, LOW level, MEDIUM-HIGH level, HIGH level, HIGH-HIGH level and alarm, LOW chlorine residual and ALARM, and HIGH chlorine residual.

1.7.9 Recycled Water Pump Station

The new recycled water pumps will convey recycled water to SFIA's recycled water users. The VFD-driven pumps will operate to maintain a pressure set point in the recycled water distribution system.

Each recycled water pump will be started and stopped at the local control station by placing its LOR switch to LOCAL and by using the START/LOCKOUT/STOP pushbuttons. Each recycled water pump will operate at the speed set point, in this mode. In REMOTE AUTO, the VFD will operate at the speed set by the SCADA system to maintain the pressure set point in the recycled water distribution system. In REMOTE MANUAL mode, the operator can adjust the speed of each recycled water pump at the VFD control panel. In the REMOTE MANUAL mode, each recycled water pump will be stopped by pushing the STOP button.

The interfaces for the recycled water pumps are:

- LOW level in the CCB/recycled water storage tank shall STOP the recycled water pumps.
- LOW chlorine residual alarm from the chlorine residual analyzer on the effluent side of the CCB/recycled water storage tank shall STOP the recycled water pumps.
- HIGH pressure switch alarm shall stop the corresponding pump.

The following alarms and parameters will be monitored for each of the RW pumps: motor ON status, REMOTE status, VFD SPEED, VFD FAIL alarm, and motor run time meter. In addition, the following signals will be monitored and displayed at the HMI: flow and pressure in the distribution system.

2.0 Preliminary Construction Cost Estimates

2.1 Construction Cost Estimate (Pending)

The construction cost opinion for the SFIA IWTP upgrade is presented in Table X (TBD), and the breakdown of cost components for each facility is presented in Appendix X (TBD). The cost opinion is based on July 2012 dollars and includes labor, materials, equipment, overhead, bonds and insurance, mobilization, and contractor's profit.

3.0 Project Schedule, and Implementation

3.1 Permitting Requirements

3.2 Schedule and Implementation

Table 3-1 provides feasible site planning options for construction and commissioning of the IWTP upgrades and tertiary treatment while keeping the IWTP operational. During the construction of all project elements, the IWTP must maintain full operational capabilities (1.7 mgd) for all units.

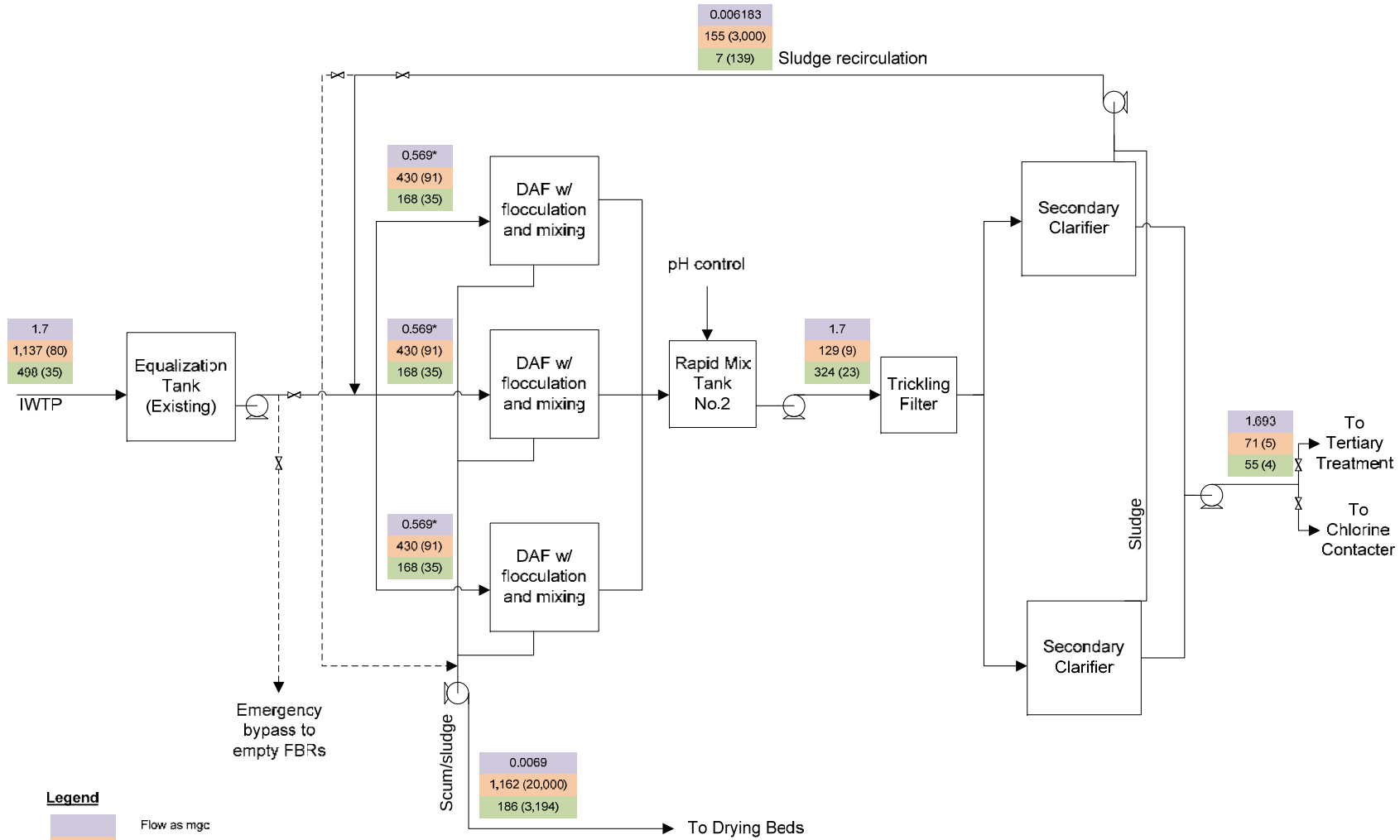
TABLE 3-1
Construction Sequencing Constraints

Component	Description/Constraints
Temporary IWTP Control Trailer	Contractor shall provide a small trailer to serve as the IWTP Control Room. This IWTP Control Room Trailer will remain in service until the new IWTP Control Room is complete *and* all remote SCADA capabilities are transferred to the new IWTP Control Room.
Trickling Filter (New)	Existing drying bed area to be demolished. A new Trickling Filter will be constructed. Upon accepted operation of the new Trickling Filter, the existing Trickling Filter will be converted to use as a Temporary Equalization Tank.
Equalization Tank	The Temporary Equalization Tank must be operational prior to taking the existing Equalization Tank out of service. The existing Equalization Tank will be refurbished or replaced.
DAFs	New DAFs, chemical storage, pump(s), and MCCs (local controls) will be constructed. Yard piping to/from new DAFs to be completed. The existing DAFs, chemical storage, pump(s), and MCCs may be demolished only after the new DAFs, chemical storage, pump(s), and MCCs are operational. Contractor must demonstrate full functionality of MCCs, including local and SCADA control, prior to demo of the existing MCCs.
Secondary Clarifiers	Demolish one of the two existing secondary clarifiers and construct a 100% Secondary Clarifier in its location. Upon acceptance of the new Secondary Clarifier, demolish the other existing Clarifier and construct a second 100% Secondary Clarifier in its location.
Tertiary Treatment (Recycled Water Facility)	Since this facility is new and requires no expansion or modification of existing facilities, it can be constructed at any time. The tertiary treatment facility shall include: Two 0.5-mgd MF/UF skids, chlorine contact basin, recycled water storage tank, sodium hypochlorite storage and feed station, recycled water pump station, and local tertiary treatment controls. SCADA Programming to SWTP Control Room (confirm) Existing recycled water facility and existing chlorine contact chamber may be demolished upon accepted operation of the new tertiary treatment facility. Piping of Secondary Clarifiers effluent to FEB.
Building 918 Remodel	This remodel cannot commence until the new Laboratory and IWTP Control Room is complete.
Building 908 Demo and New Building	This building cannot be demolished until the following is built and accepted: - New DAFs, chemical storage, and pump - All MCCs functionality currently housed in the building are rebuilt and operational - All IWTP Control Room Functionality is available in the IWTP Control Room Trailer
Bioassay Trailer Removal	Bioassay trailer shall be removed following transfer of bioassay equipment to new lab.

Based on the constraints shown during construction, a preliminary schedule was developed and is presented in Appendix D.

Appendix A
Process Schematics and Mass Balances

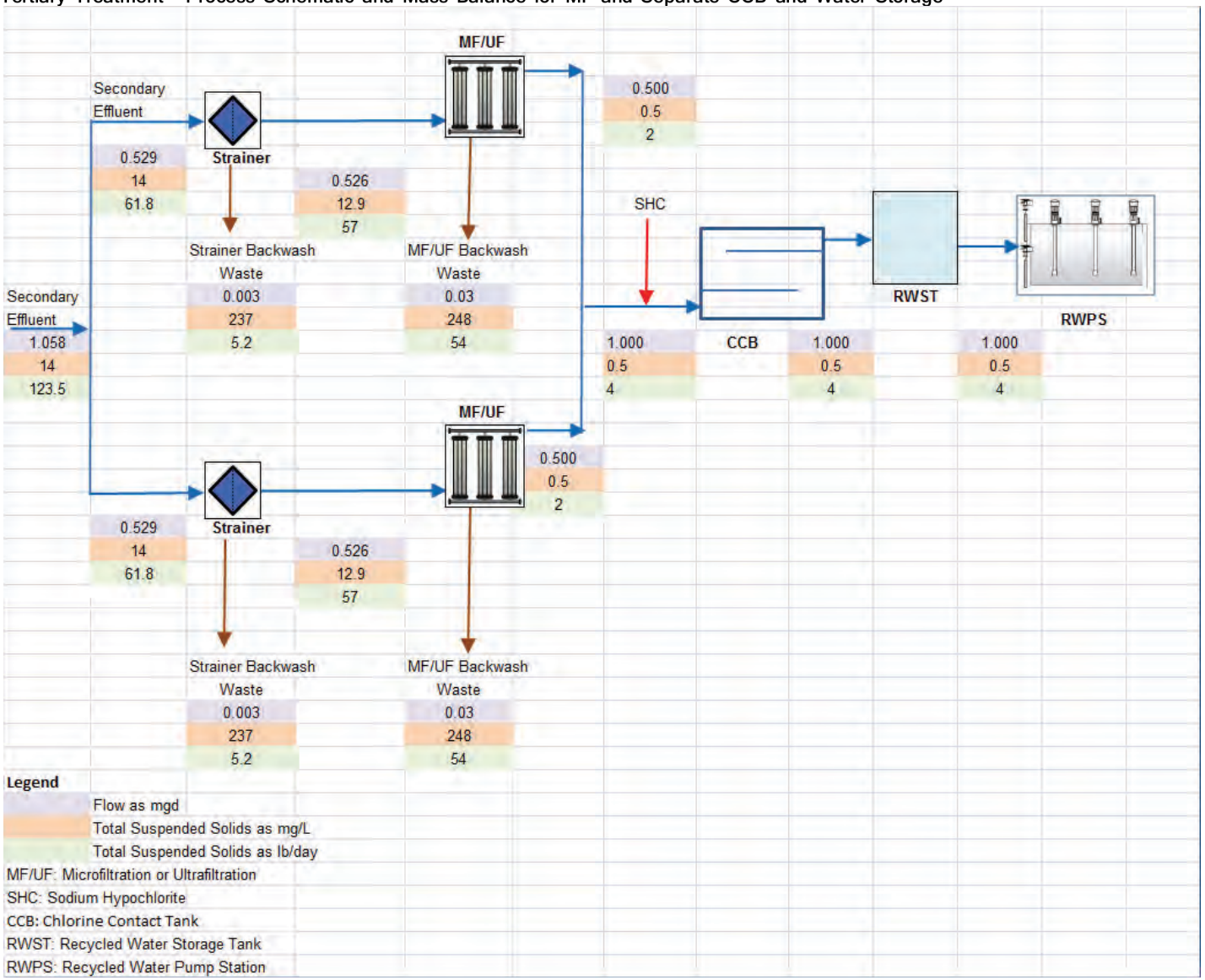
Primary and Secondary Treatment - Process Schematic and Mass Balance



*The mass balance shows an equal flow distribution to the three DAF units.
However the DAF units will be sized for 50% capacity, equivalent to 0.85 mgd.

WBG090512211739SCO

Tertiary Treatment - Process Schematic and Mass Balance for MF and Separate CCB and Water Storage



Legend

Flow as mgd

Total Suspended Solids as mg/L

Total Suspended Solids as lb/day

MF/UF: Microfiltration or Ultrafiltration

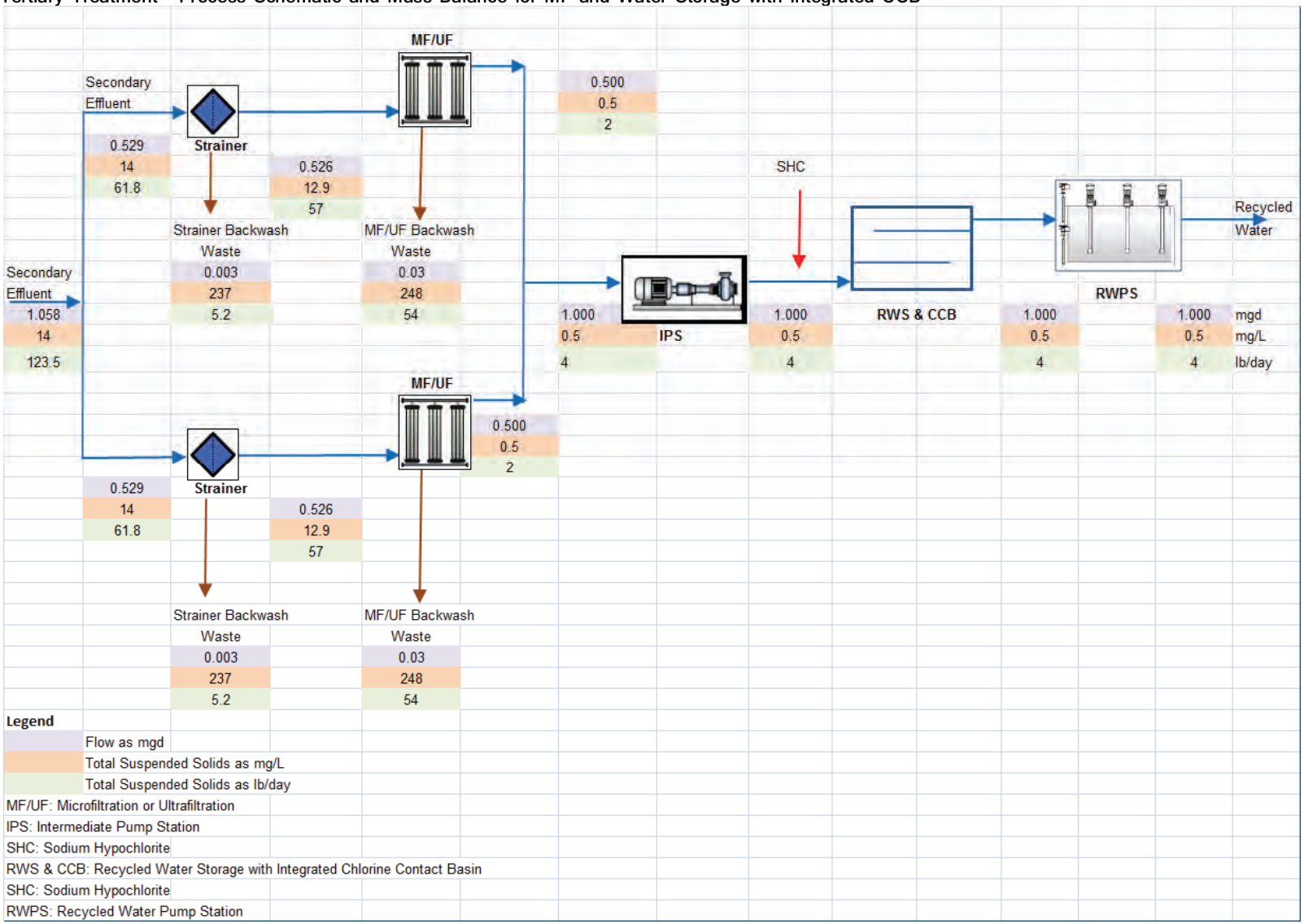
SHC: Sodium Hypochlorite

CCB: Chlorine Contact Tank

RWST: Recycled Water Storage Tank

RWPS: Recycled Water Pump Station

Tertiary Treatment - Process Schematic and Mass Balance for MF and Water Storage with Integrated CCB



Appendix B
Vendor Quotes

From: An, Li [mailto:li.an@veoliawater.com]
Sent: Tuesday, August 07, 2012 8:57 AM
To: Abercrombie, Laure/DEN
Cc: Grigorieff, Mike/SCO
Subject: RE: CH2M Hill - DAF quote

Hello Laure,

The scope (as shown on attached drawings) for 3x Circular DAFs, each with skid-mounted air dissolving tube and Recycle Pump, each with Support Stand with Access platform, each with Tube Flocculator with Coagulant Static Mixer. A single NEMA-4 PLC-based Control Panel will control all three DAFs.

Total Budgetary Price is $775,000.

Please let me know if you need anything else.

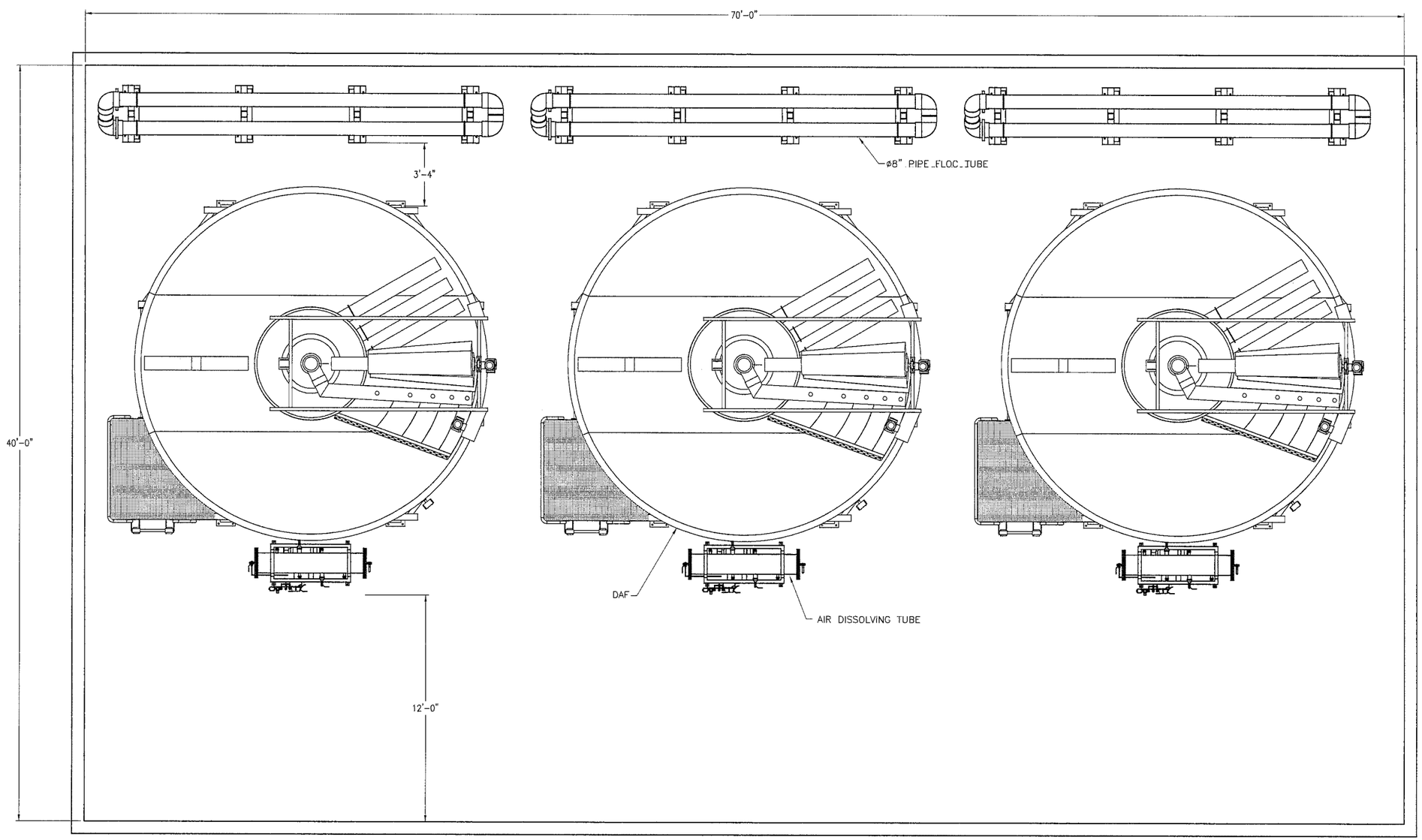
Thank you,

Li An

Li An, PhD
Senior Application Engineer

N.A. Water Systems
A Veolia Water Solutions & Technologies Company
Airside Business Park
250 Airside Drive
Moon Township, PA 15108
Direct: 412-809-6673
Mobile: 724-719-0130
Facsimile: 412-809-6075
Email: li.an@veoliawater.com
www.nawatersystems.com
Service ◦ Value ◦ Responsibility

PLEASE CONSIDER THE ENVIRONMENT BEFORE PRINTING THIS EMAIL



PLAN VIEW

REV. NO.	ISSUE DATE	DRAWN BY	CHECKED BY	REVIEWED BY	APPROVED BY	APPROVED BY (OPTIONAL)	DESCRIPTION
A	8/7/12	B. MCGINNIS	L. AN				PRELIMINARY

DESIGN/DRAFTING — ENGINEERING

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N.A. WATER SYSTEMS

TEL 800-337-0777
250 AIRSIDE DRIVE
MOON TOWNSHIP, PA 15108-2783
PROCESS SOLUTIONS FOR THE WATER INDUSTRY

VEOLIA WATER
Solutions & Technologies

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SCALE: 3/8"= 1'-0"

DRAWN BY: B. MCGINNIS

TITLE: CH2M HILL

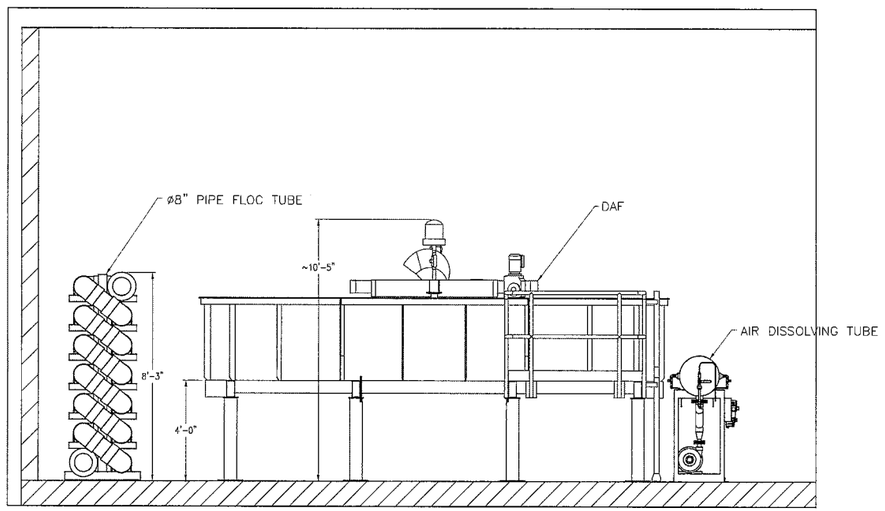
DAFS (3 UNITS)
GENERAL ARRANGEMENT

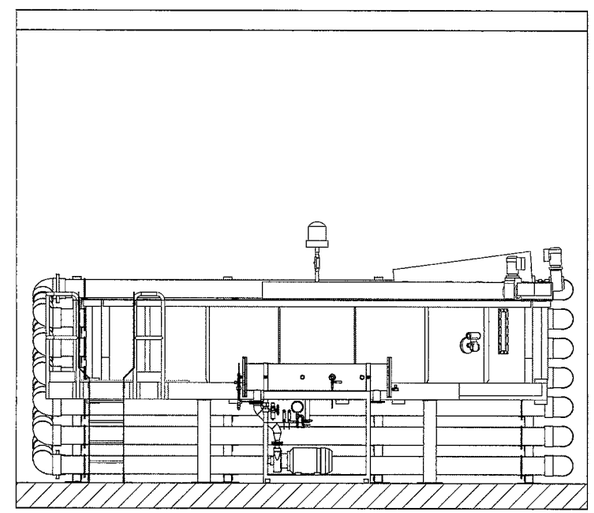
CONTRACT NO.

DWG. NO. M101-01

REV A

08/07/2012 11:34:19 AM C:\Local_Workspace\AutoCAD\5600119328\5600119328_M101-01.dwg





ELEVATION VIEWS

REV. NO.	ISSUE DATE	DRAWN BY	CHECKED BY	REVIEWED BY	APPROVED BY	APPROVED BY (OPTIONAL)	DESCRIPTION
A	8/7/12	B. MCGINNIS	L. AN				PRELIMINARY

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SCALE: 3/8"= 1'-0"
DRAWN BY: B. MCGINNIS
TITLE: CH2M HILL

DAFS (3 UNITS)
ELEVATION VIEWS

CONTRACT NO.:
DWG. NO.: M101-02
REV: A

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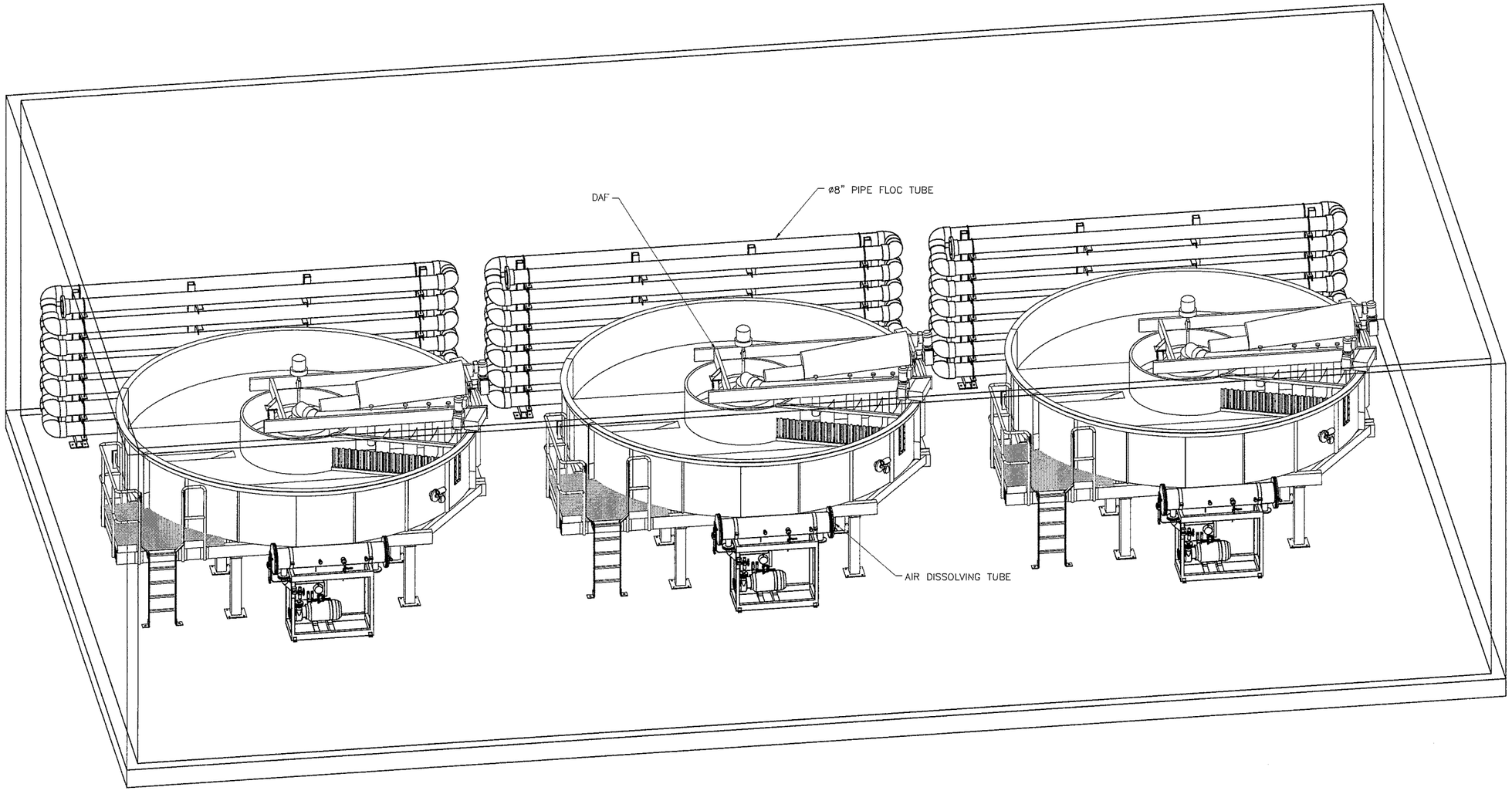

ISO–VIEW

REV. NO.	ISSUE DATE	DRAWN BY	CHECKED BY	REVIEWED BY	APPROVED BY	APPROVED BY (OPTIONAL)	DESCRIPTION
A	8/7/12	B. MCGINNIS	L. AN				PRELIMINARY

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SCALE: 3/8"= 1'-0"

DRAWN BY: B. MCGINNIS

TITLE: CH2M HILL

DAFS (3 UNITS)
ISO VIEW

CONTRACT NO.

DWG. NO. M101-03

REV. A

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WesTech
Experts in Liquid/Solids Separation

Budget Proposal For:
CH2M, CA

Engineer:
CH2M HILL

Equipment:
Rectangular Dissolved Air Flotation Units

Represented By:
Sepco, Inc.
1224 Centennial Road
Fort Collins, CO 80525
Contact: Dan Bruce
Phone: (970) 282-9015
Fax: (970) 282-9020
danbruce.sepco@earthlink.net

Furnished By:
WesTech Engineering, Inc.
Salt Lake City, Utah
Contact: Michael Vanderhooft / Ian Fife
Phone: (801) 265-1000
email: mvanderhooft@westech-inc.com
ifife@westech-inc.com

WesTech Proposal: 1210403 rev 1
Date: August 8, 2012

BUDGET PRICING

Unless otherwise indicated, prices listed below are for equipment only.

ITEM	EQUIPMENT	PRICE (U.S. $)
A	Three (3) 8' Wide x 58' Long Rectangular Dissolved Air Flotation Units with Saturation Systems and V-Bottom Steel Tanks. WesTech Model DAFR6S	$ 735,000.00
Option 1	Three (3) 46' long screw conveyors.	$ 63,500.00
Option 2	Three (3) air compressors	$ 12,600.00

EQUIPMENT DESCRIPTION

ITEM A: Three (3) 8' Wide x 58' Long Rectangular Dissolved Air Flotation Units with Saturation Systems and V-Bottom Steel Tanks. WesTech Model DAFR6S

General

Each DAF unit is designed for an influent flow rate of 590 gpm (0.85 mgd). The influent TSS design is for a maximum of 400 mg/L. The recycle rate is 150 gpm, resulting in a hydraulic loading rate of rate of 2.0 gpm/ft^2 and an air to solids ratio greater than 0.02 lbs air/lbs solids.

DESIGN FEATURES

The system shall be designed to recirculate a portion of the clean effluent through a pressurization system. This pressurization system shall saturate the recycle with pressurized air and then inject the mixed solution into the DAF tank at the influent point of entry. Introduction of the air saturated liquid, influent feed, and the collection of floated solids shall be all accomplished in a single tank.

Recirculation of a percentage of the effluent shall be accomplished by means of a centrifugal recycle pump. The pump shall propel flow into a pressurization vessel (retention tank) where air shall be introduced and the two mixed under pressure to produce an air saturated liquid. The resultant flow shall then pass out of a distribution manifold where it will thoroughly mix with the influent flow. Solids that adhere to the air particles will float and be removed by skimmer assemblies. The float solids will be elevated up a sloped beach surface and dropped into the float box where it will then be removed from the DAF tank. Any solids that settle to the bottom of the tank shall be transferred to the end of the tank by a bottom screw conveyor assembly for periodically removed by a manually actuated valve.

The clarified effluent will flow under a float baffle separating the DAF cell from an adjustable weir. The clean effluent will fill a small clear well where the flow is split between the recycle and the clarified water.

EACH UNIT FURNISHED COMPLETE WITH THE FOLLOWING COMPONENTS:

- One (1) 8' Wide x 58' long (48' for the floatation zone) v-bottom steel tank fabricated of carbon steel plate with suitable reinforcements.

- One (1) 8' Wide x 7' long Flocculation basin with flocculating mixer. Flocculation basin has an approximate detention of 5 minutes at the design flow rate.

- One (1) 3' Wide x 3' long flash mix basin with mixer. Flash mix basin has an approximate detention of 1 minute at the design flow rate.

- One (1) 1.5' long clearwell integral to the tank

- One (1) skimmer drive unit directly coupled to the drive shaft with a VFD rated ½ hp 230/460 VAC, 3 phase, TEFC motor.
- Two (2) shafts, drive and tail, of carbon steel with bearings.
- One (1) carbon steel effluent baffle with adjustable overflow weir integral to the tank.
- One (1) carbon steel effluent clear well integral to the tank.
- Four (4) non-metallic sprockets with non-metallic chain.
- Nineteen (19) FRP flight skimmer assemblies with wipers.
- One (1) carbon steel float beach and float box combination integral to the tank.
- One (1) 3" backpressure control valve Haymore style with horizontal section of pipe.
- One (1) bottom solids auger to transport settled solids to sludge hopper.
- One (1) auger drive unit directly coupled to the auger drive shaft with a ½ hp 230/460 VAC, 3 phase, TEFC motor.
- One (1) saturation tank, 30” diameter x 48” side shell height, with level controls, sight glass, pressure relief valve and pressure gauge.
- One (1) stainless steel air control panel with air flow control, solenoid valve, rotameter, check valve, filter regulator and pressure gauge.
- One (1) 15 hp horizontal centrifugal recycle pump, rated for 150 gpm at 175' TDH with 230/460 VAC, 3 phase, TEFC motor.
- Plant air will be used, and should be able to supply 2.5 SCFM at 100 psi or greater, otherwise a dedicated air compressor will be needed.
- One (1) electric control panel for control of proposed equipment.
- One (1) lot 304 stainless steel hardware and fasteners for all supplied equipment.
- Ferrous metal will be blasted with one (1) coat of epoxy primer and one (1) coat of epoxy top coat.
- Field service of two (2) trips for four (4) days for inspection, start-up and training.

Option 1: Three (3) 46' Long screw conveyors. (One for each DAF unit)

- Each DAF unit will be supplied with one (1) rotary screw coveyor system complete with Carbon Steel auger, electrical motor, reducer, and bearings.

- The DAF tanks will be modified with a half pipe bottom to accommodate the screw conveyor, as well as mechanical seals.

Option 2: Three (3) air compressors. (One for each DAF unit)

- Each DAF unit will be supplied with one (1) reciprocating air compressor with compressor receiving tank. Each compressor is capable of supplying 6.7 SCFM of air at 100 psi into the receiving tank.

NOTE: ANY ITEM NOT LISTED ABOVE TO BE FURNISHED BY OTHERS.

These proposal sections have been reviewed for accuracy and approved for issue:

By: *Michael Vanderhooft* Date: August 8, 2012

WESTECH TERMS

Unless otherwise indicated, prices listed are for equipment only. Prices are based on the estimated cost of goods and services at the budget proposal date. Future escalation is not included in the pricing.

Pricing is based on fabricating, assembling, inspecting, delivering and performing field service on multiple units concurrently. If any of these phases are to be broken out into separate occurrences upon customer request, a change order may be required for the additional material and/or resources this extra process may incur.

Sales Tax: No sales taxes, use taxes, or duties have been included in our pricing.

Equipment Payment Terms: Terms for equipment are 15 percent payment of the purchase price with submittal drawings, 35 percent upon release to fabrication, and 50 percent net 30 days from shipment.

Freight: Not Quoted.

Schedule: Approval drawings will be submitted within 6 to 8 weeks after receipt and acceptance of purchase order.

Shipment: Estimated shipment time is 16 to 18 weeks after approved shop drawings are received in our office.

WARRANTY

WesTech equipment is backed by WesTech's reputation as a quality manufacturer, and by many years of experience in the design of reliable equipment.

Equipment manufactured or sold by WesTech Engineering, Inc., once paid for in full, is backed by the following warranty:

For the benefit of the original user, WesTech warrants all new equipment manufactured by WesTech Engineering, Inc. to be free from defects in material and workmanship, and will replace or repair, F.O.B. its factories or other location designated by it, any part or parts returned to it which WesTech's examination shall show to have failed under normal use and service by the original user within one (1) year following initial start-up, or eighteen (18) months from shipment to the purchaser, whichever occurs first. Such repair or replacement shall be free of charge for all items except for those items such as resin, filter media and the like that are consumable and normally replaced during maintenance, with respect to which, repair or replacement shall be subject to a pro-rata charge based upon WesTech's estimate of the percentage of normal service life realized from the part. WesTech's obligation under this warranty is conditioned upon its receiving prompt notice of claimed defects, which shall in no event be later than thirty (30) days following expiration of the warranty period, and is limited to repair or replacement as aforesaid.

THIS WARRANTY IS EXPRESSLY MADE BY WESTECH AND ACCEPTED BY PURCHASER IN LIEU OF ALL OTHER WARRANTIES, INCLUDING WARRANTIES OF MERCHANTABILITY AND FITNESS FOR PARTICULAR PURPOSE, WHETHER WRITTEN, ORAL, EXPRESS, IMPLIED, OR STATUTORY. WESTECH NEITHER ASSUMES NOR AUTHORIZES ANY OTHER PERSON TO ASSUME FOR IT ANY OTHER LIABILITY WITH RESPECT TO ITS EQUIPMENT. WESTECH SHALL NOT BE LIABLE FOR NORMAL WEAR AND TEAR, CORROSION, OR ANY CONTINGENT, INCIDENTAL, OR CONSEQUENTIAL DAMAGE OR EXPENSE DUE TO PARTIAL OR COMPLETE INOPERABILITY OF ITS EQUIPMENT FOR ANY REASON WHATSOEVER.

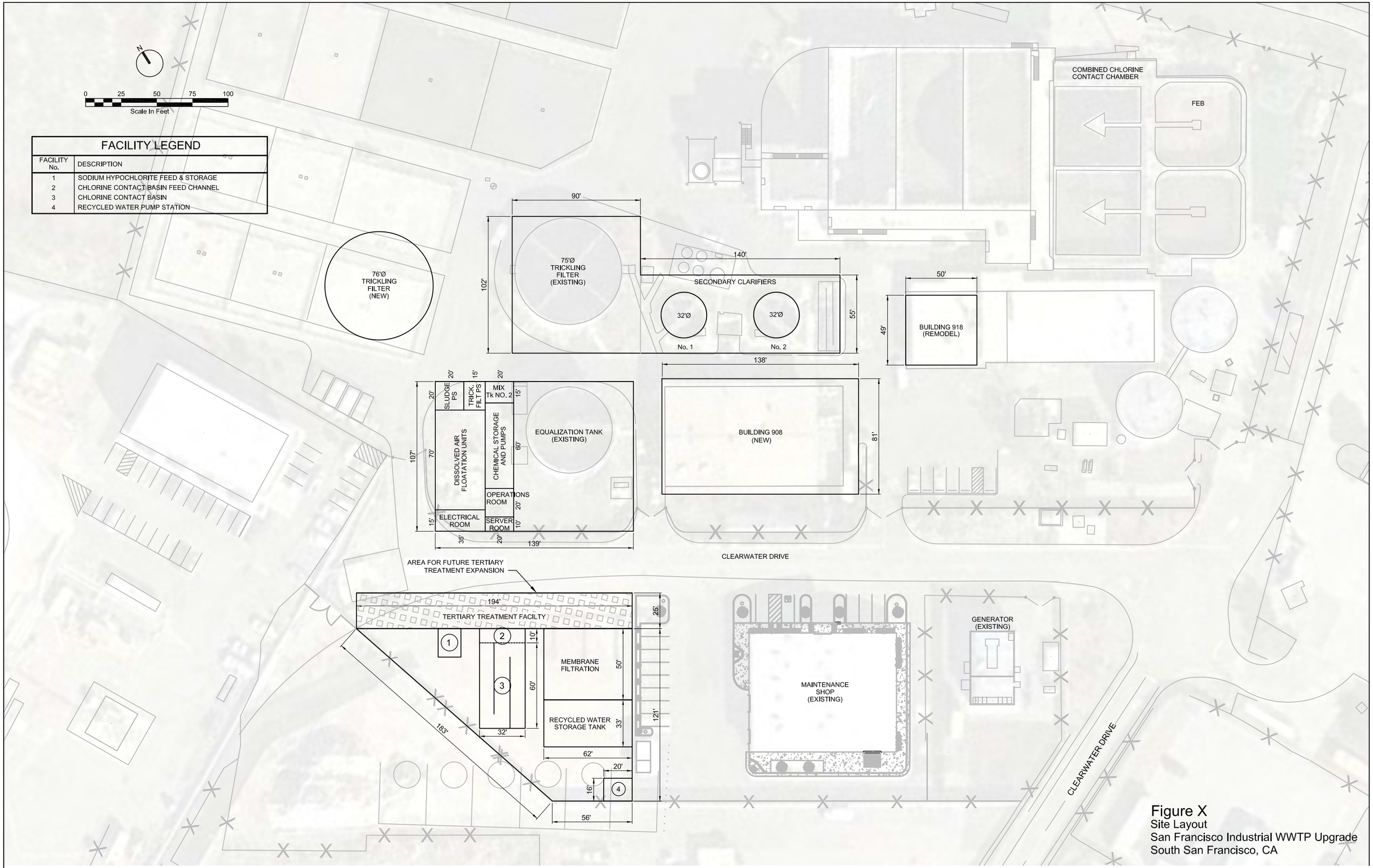
This warranty shall not apply to equipment or parts thereof which have been altered or repaired outside of a WesTech factory, or damaged by improper installation, application, or maintenance, or subjected to misuse, abuse, neglect, accident, or incomplete adherence to all manufacturer’s requirements, including, but not limited to, Operations & Maintenance Manual guidelines & procedures.

This warranty applies only to equipment made or sold by WesTech Engineering, Inc.

WesTech Engineering, Inc. makes no warranty with respect to parts, accessories, or components purchased by the customer from others. The warranties which apply to such items are those offered by their respective manufacturers.

QF-00-032E Rev. 08/18/05

Appendix C
Preliminary Layout



FACILITY LEGEND

FACILITY No.	DESCRIPTION
1	SODIUM HYPOCHLORITE FEED & STORAGE
2	CHLORINE CONTACT BASIN FEED CHANNEL
3	CHLORINE CONTACT BASIN
4	RECYCLED WATER PUMP STATION

Figure X
Site Layout
San Francisco Industrial WWTP Upgrade
South San Francisco, CA

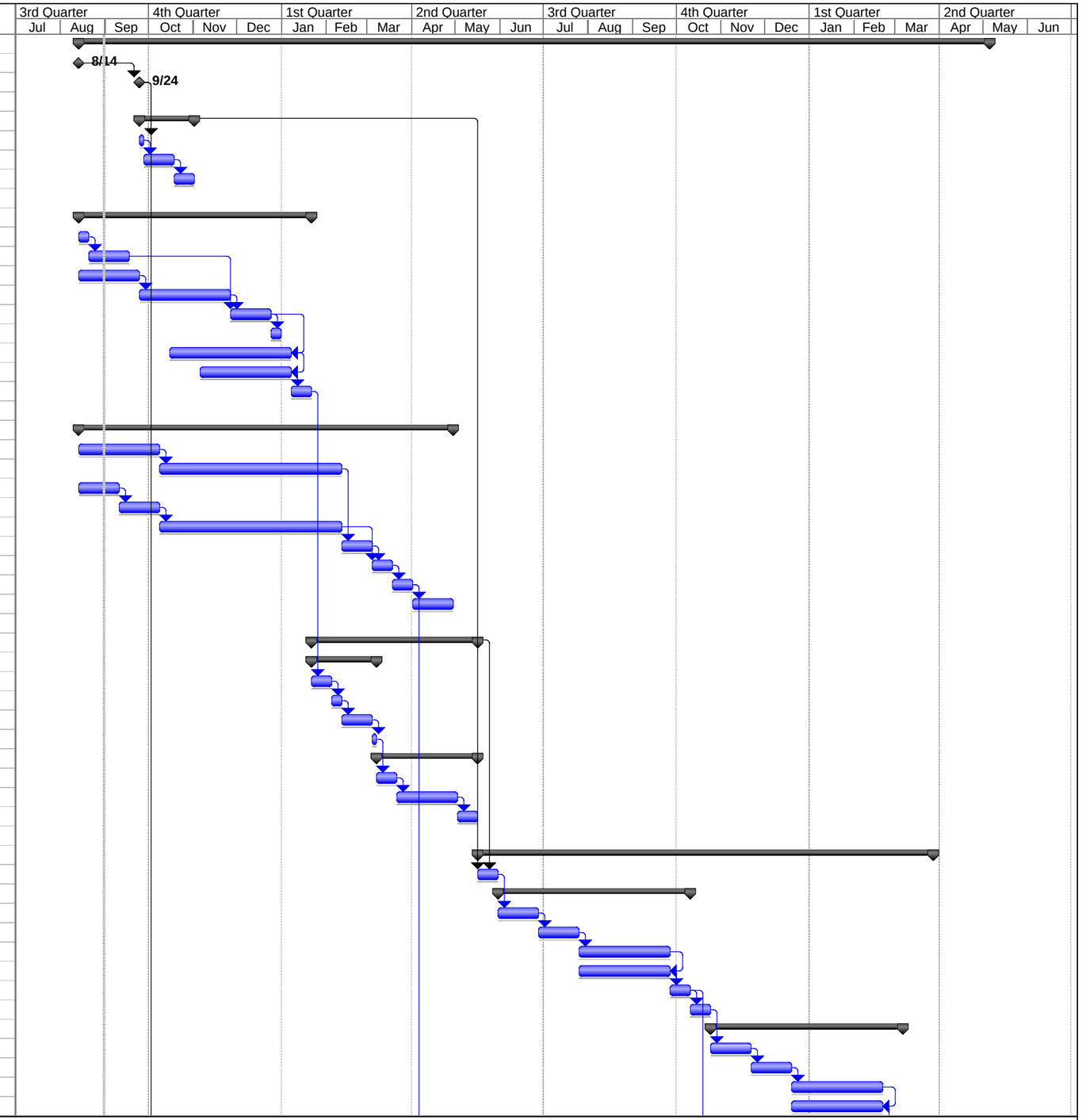
PROJECT-NO

SOURCE: 2012 TERRAMETRICS - IMAGERY DATE: 10-31-2011

CH2MHILL®

Appendix D
Preliminary Construction Schedule

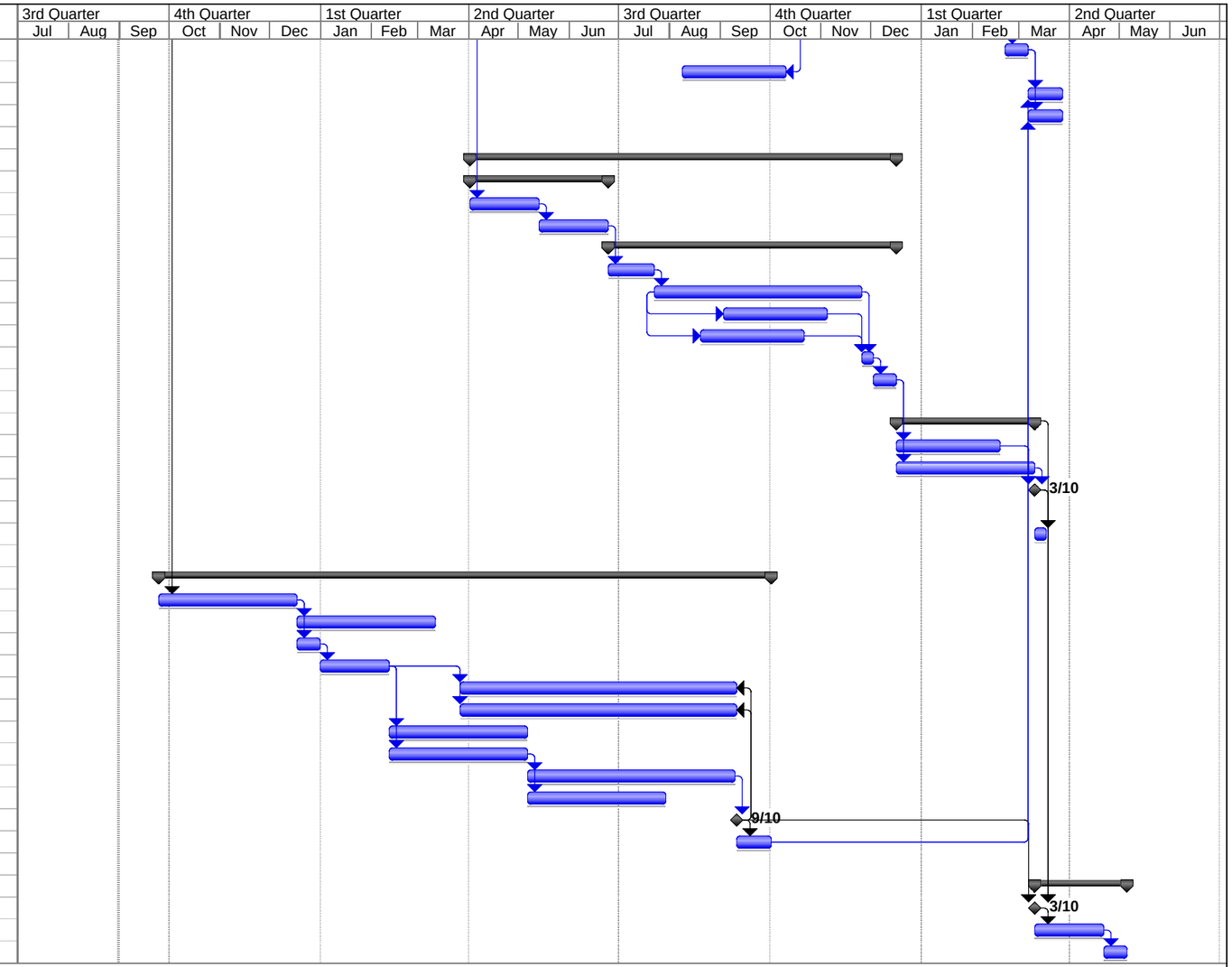
ID	Task Name	Duration	Start	Finish	Predecessors
1	**SFIA Industrial WWTP Project**	**450 days**	**Tue 8/14/12**	**Mon 5/5/14**	
2	Notice of Award	0 days	Tue 8/14/12	Tue 8/14/12	
3	Notice to Proceed	30 days	Tue 8/14/12	Mon 9/24/12	2
4					
5	**Temporary IWTP Trailer**	**28 days**	**Tue 9/25/12**	**Thu 11/1/12**	
6	Mobilize and install temp trailer	3 days	Tue 9/25/12	Thu 9/27/12	3
7	Configure temp IWTP trailer	15 days	Fri 9/28/12	Thu 10/18/12	6
8	Functional Testing and Acceptance	10 days	Fri 10/19/12	Thu 11/1/12	7
9					
10	**Trickling Filter (New)**	**115 days**	**Tue 8/14/12**	**Mon 1/21/13**	
11	Demo Existing Drying Bed and prep site	5 days	Tue 8/14/12	Mon 8/20/12	
12	Piling/Foundation	20 days	Tue 8/21/12	Mon 9/17/12	11
13	TF Shop drawings submttal and approval	30 days	Tue 8/14/12	Mon 9/24/12	
14	Fabrication and delivery	45 days	Tue 9/25/12	Mon 11/26/12	13
15	Steel Tank Construction	20 days	Tue 11/27/12	Mon 12/24/12	14,12
16	Install media	5 days	Tue 12/25/12	Mon 12/31/12	15
17	Install TF Equip., mechanical piping, pumps, and electircal	60 days	Tue 10/16/12	Mon 1/7/13	15FF+10 days
18	Electrical and I&C	45 days	Tue 11/6/12	Mon 1/7/13	17FF
19	Trickling Filter Acceptance Testing	10 days	Tue 1/8/13	Mon 1/21/13	18
20					
21	**New DAFs, Chem Storage, Pumps, and MCC**	**185 days**	**Tue 8/14/12**	**Mon 4/29/13**	
22	DAF Shop drwgs submittal and approval	40 days	Tue 8/14/12	Mon 10/8/12	
23	DAF Fabrication and delivery	90 days	Tue 10/9/12	Mon 2/11/13	22
24	Site Demo and Preparation	20 days	Tue 8/14/12	Mon 9/10/12	
25	Foundation	20 days	Tue 9/11/12	Mon 10/8/12	24
26	Chemical Storage/Pump Room, incl mech and elec.	90 days	Tue 10/9/12	Mon 2/11/13	25
27	DAF Units installed	15 days	Tue 2/12/13	Mon 3/4/13	23
28	Local IWTP Controls (MCCs) connected and tested	10 days	Tue 3/5/13	Mon 3/18/13	27,26
29	Testing and Acceptance of DAF Units	10 days	Tue 3/19/13	Mon 4/1/13	28
30	Demo exst DAFs, chem storage, pumps, and MCC	20 days	Tue 4/2/13	Mon 4/29/13	29
31					
32	**Equalization Tank (Refurb/Replace)**	**83 days**	**Tue 1/22/13**	**Thu 5/16/13**	
33	**Existing Trickling Filter Converted to Temp EQ Tank**	**33 days**	**Tue 1/22/13**	**Thu 3/7/13**	
34	Remove exst TF equipment	10 days	Tue 1/22/13	Mon 2/4/13	19
35	Remove exst TF Media	5 days	Tue 2/5/13	Mon 2/11/13	34
36	Install temporary pumps and piping for DAF process	15 days	Tue 2/12/13	Mon 3/4/13	35
37	Testing and Acceptacne of Temp EQ Tank	3 days	Tue 3/5/13	Thu 3/7/13	36
38	**Refurbish Existing Equalization Tank**	**50 days**	**Fri 3/8/13**	**Thu 5/16/13**	
39	Remove equip and prep	10 days	Fri 3/8/13	Thu 3/21/13	37
40	Repair/replace equip and install	30 days	Fri 3/22/13	Thu 5/2/13	39
41	Testing and Accpetance of Refurbished EQ Tank	10 days	Fri 5/3/13	Thu 5/16/13	40
42					
43	**Secondary Clarifiers**	**225 days**	**Fri 5/17/13**	**Thu 3/27/14**	
44	Demo Exst Clarifier No.1	10 days	Fri 5/17/13	Thu 5/30/13	5,32
45	**Construct new Secondary Clarifier No.1**	**95 days**	**Fri 5/31/13**	**Thu 10/10/13**	
46	Foundation	20 days	Fri 5/31/13	Thu 6/27/13	44
47	Construct Tank - 32' dia x 15' high	20 days	Fri 6/28/13	Thu 7/25/13	46
48	Install equipment, and mech piping	45 days	Fri 7/26/13	Thu 9/26/13	47
49	Electrical and I&C	45 days	Fri 7/26/13	Thu 9/26/13	48FF
50	Testing and Acceptance	10 days	Fri 9/27/13	Thu 10/10/13	49
51	Demo Exst Clarifire No.2	10 days	Fri 10/11/13	Thu 10/24/13	50
52	**Construct new Secondary Clarifier No.2**	**95 days**	**Fri 10/25/13**	**Thu 3/6/14**	
53	Foundation	20 days	Fri 10/25/13	Thu 11/21/13	51
54	Construct Tank - 32' dia x 15' high	20 days	Fri 11/22/13	Thu 12/19/13	53
55	Install equipment, piping, electrical	45 days	Fri 12/20/13	Thu 2/20/14	54
56	Electrical and I&C	45 days	Fri 12/20/13	Thu 2/20/14	55FF



Project: SFIA Industrial WWTP
Date: Fri 8/31/12

Task | Split | Progress | Milestone | Summary | Project Summary | External Tasks | External Milestone | Deadline

ID	Task Name	Duration	Start	Finish	Predecessors
57	Testing and Acceptance	10 days	Fri 2/21/14	Thu 3/6/14	56
58	Yard piping Secondary Clarifier effluent to FEB	45 days	Fri 8/9/13	Thu 10/10/13	50FF
59	Demolish existing Chlorine Contact Chamber	15 days	Fri 3/7/14	Thu 3/27/14	93,57
60	Demolish existing RW Facility	15 days	Fri 3/7/14	Thu 3/27/14	93,57
61					
62	**Building 908**	**185 days**	**Tue 4/2/13**	**Mon 12/16/13**	
63	**Building 908 Demo**	**60 days**	**Tue 4/2/13**	**Mon 6/24/13**	
64	Remove equipment, salvage materials	30 days	Tue 4/2/13	Mon 5/13/13	29
65	Demo building structure and prep site	30 days	Tue 5/14/13	Mon 6/24/13	64
66	**Building 908 New**	**125 days**	**Tue 6/25/13**	**Mon 12/16/13**	
67	Foundation	20 days	Tue 6/25/13	Mon 7/22/13	65
68	Building Structure	90 days	Tue 7/23/13	Mon 11/25/13	67
69	Install equipment	45 days	Tue 9/3/13	Mon 11/4/13	68SS+30 days
70	Install mechanical and electrical	45 days	Tue 8/20/13	Mon 10/21/13	68SS+20 days
71	All IWTP MCC and SCADA Controls accepted	5 days	Tue 11/26/13	Mon 12/2/13	69,70,68
72	Testing and Acceptance	10 days	Tue 12/3/13	Mon 12/16/13	71
73					
74	**Building 918 Remodel**	**60 days**	**Tue 12/17/13**	**Mon 3/10/14**	
75	Lunch rooom / conference room	45 days	Tue 12/17/13	Mon 2/17/14	72
76	Restrooms and Locker Facilities (men and women)	60 days	Tue 12/17/13	Mon 3/10/14	72
77	Complete remodel	0 days	Mon 3/10/14	Mon 3/10/14	75,76
78					
79	Bioassay Trailer Removal	5 days	Tue 3/11/14	Mon 3/17/14	77
80					
81	**Tertiary Treatment (Recyled Water Facility)**	**266 days**	**Tue 9/25/12**	**Tue 10/1/13**	
82	Shop drawings submittal and approval	60 days	Tue 9/25/12	Mon 12/17/12	3
83	Fabrication and delivery	60 days	Tue 12/18/12	Mon 3/11/13	82
84	Site clearing and Preparation	10 days	Tue 12/18/12	Mon 12/31/12	82
85	Foundations	30 days	Tue 1/1/13	Mon 2/11/13	84
86	Mechanical piping	120 days	Wed 3/27/13	Tue 9/10/13	92FF,85
87	Electrical, I&C	120 days	Wed 3/27/13	Tue 9/10/13	92FF,85
88	Recycled Water Pump Station	60 days	Tue 2/12/13	Mon 5/6/13	85
89	Chlorine Contact Basins and Feed Channel	60 days	Tue 2/12/13	Mon 5/6/13	85
90	Membrane Filtration Facility	90 days	Tue 5/7/13	Mon 9/9/13	89
91	Sodium Hypochlorite Feed and Storage	60 days	Tue 5/7/13	Mon 7/29/13	89
92	Tertiary Treatment - ready for testing	1 day	Tue 9/10/13	Tue 9/10/13	90
93	Testing and Acceptance	15 days	Wed 9/11/13	Tue 10/1/13	92
94					
95	**Contractract Closeout**	**40 days**	**Mon 3/10/14**	**Mon 5/5/14**	
96	Contract ready for closeout	0 days	Mon 3/10/14	Mon 3/10/14	74,77,92
97	Final punch list and acceptance	30 days	Tue 3/11/14	Mon 4/21/14	96
98	Final Acceptance and Contractor demobilization	10 days	Tue 4/22/14	Mon 5/5/14	97



Project: SFIA Industiral WWTP
Date: Fri 8/31/12

Task | Progress | Summary | External Tasks | Deadline
Split | Milestone | Project Summary | External Milestone

Appendix E
Previous Technical Memorandums

Appendix E.2
Technical Memorandum 2

San Francisco International Airport Industrial Wastewater Treatment Plant Data Analysis

PREPARED FOR: San Francisco International Airport

PREPARED BY: CH2M HILL

DATE: March 20, 2012

Introduction and Objectives

The City and County of San Francisco owns and operates the Mel Leong Treatment Plant. The Industrial Wastewater Treatment Plant (IWTP) treats industrial wastewater as well as first flush, storm water runoff from airport facilities. The IWTP is comprised of flow equalization, rapid mix and flocculation, dissolved air floatation, trickling filter, secondary clarification, and chlorine based disinfection. The effluent form the IWTP is routed to either the effluent pump station or the water reclamation pump station. The combined effluent is pumped to the North Bayside System Unit outfall, where the treated water is combined with effluent from South San Francisco, Millbrae, and Burlingame. Dechlorination takes place in the shared outfall before the effluent is discharged in to the Bay. The effluent from the water reclamation pump station is used within the Airport for irrigation purposes under a restricted use permit. The sludge and scum collected from dissolved air floatation and secondary clarification are conveyed to sludge drying beds for dewatering and drying.

The IWTP collection system collects wastewater from point sources from specific areas (maintenance shops, etc) and surface runoff. The point source wastewater is collected by the industrial wastewater collection system and pumped to the IWTP. The first part of runoff (first flush) is collected by first flush ponds. The collected runoff is then pumped to the IWTP at a controlled rate to avoid shock loading the IWTP.

In this technical memorandum (TM), historical wastewater treatment flow and water quality data (from 2006 to 2010) for the IWTP as well as Sanitary Wastewater Treatment Plant (SWTP) are analyzed. The purpose of this technical memorandum is to analyze the existing flow and water quality data to characterize raw and treated wastewater. These data will be a basis for establishing design criteria and selecting treatment unit processes to produce disinfected tertiary recycled water for unrestricted irrigation, urban reuse and industrial reuse purposes.

Evaluation of Historical IWTP Flows and Water Quality Data

The flow data between 2006 and 2010 were analyzed to establish:

- **annual average daily flow** - average of flow that occurs during a specified period (total flow during a specified period is divided by the number of days in the specified period). If the period covers an entire year, it refers to annual average day flow)
- **maximum monthly flow (Qmax 30)** – the largest of flow anticipated to occur during a continuous 30-day period, expressed as daily average
- **maximum day flow** - The highest flow anticipated to occur during a 1-day period, expressed as a daily average
- **peak instantaneous flow-** The highest flow anticipated to occur during a short period (i.e., 30 minutes), expressed as daily average
- **minimum day flow** - The lowest flow anticipated to occur during a 1-day period, expressed as a daily average

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Because of the lack of hourly and instantaneous flows data, these flows could not be established. However, these flows are not critical in our evaluation knowing that the existing ponds and equalization tank dampen the diurnal peak flows occurring at the IWTP.

Average day flow is important for estimating annual operation and maintenance (O&M) cost while maximum month flow (Qmax 30) is important for sizing treatment facilities.

The analyzed data indicated that the IWTP has received an average annual day flow of 0.58 mgd between the data evaluation period. During the same period, minimum day, maximum monthly flow and maximum day flows were 0.11, 0.63 and 1.62 mgd, respectively. Approximately 90% of the daily flows were less than 1.0 mgd during the evaluation period. Table 1 summarizes the IWTP flows and established flow ratios. The detailed flow analysis is presented in Appendix A.

Table 1 IWTP Flows and Flow Ratios (2006-2010)

Parameter	Value
Annual Average Day Flow, mgd	0.58
Maximum Monthly Flow (Qmax30), mgd	0.63
Minimum Day Flow, mgd	0.11
Maximum Day Flow, mgd	1.62
Peak Instantaneous Flow, mgd	2.58
Maximum Month to Annual Average Flow Ratio	1.09
Minimum Day to Annual Average Day Flow Ratio	0.19
Maximum Day to Annual Average Day Flow Ratio	2.79

The flows presented in Table 1 indicate significant fluctuations as minimum, annual average and maximum day flows are compared. Seasonal storm events and fluctuations in point source waste generation are the main reasons for such fluctuations. Although such fluctuations are quite normal, the facility design should have flexibility to handle variations in wastewater flows and loads without sacrificing plant performance.

Secondary effluent (treated wastewater) from the Sanitary Wastewater Treatment Plant (SWTP) may be blended with IWTP treated flow prior to tertiary treatment to increase reclaimed water production capacity. The data evaluation in this TM also covered Sanitary Treatment Plant effluent flows and key water quality parameters for treated effluent. Table 2 summarizes flows between 2007 and 2011 for the SWTP. The detailed flow data for the SWTP are presented in Appendix B.

Table 2 SWTP Effluent Flows (2007-2011)

Parameter	Value
Annual Average Day Flow, mgd	0.58
Maximum Monthly Flow (Qmax30), mgd	0.70
Minimum Day Flow, mgd	0.34
Maximum Day Flow, mgd	1.05
Maximum Month to Annual Average Flow Ratio	1.21
Minimum Day to Annual Average Day Flow Ratio	0.59
Maximum Day to Annual Average Day Flow Ratio	1.81

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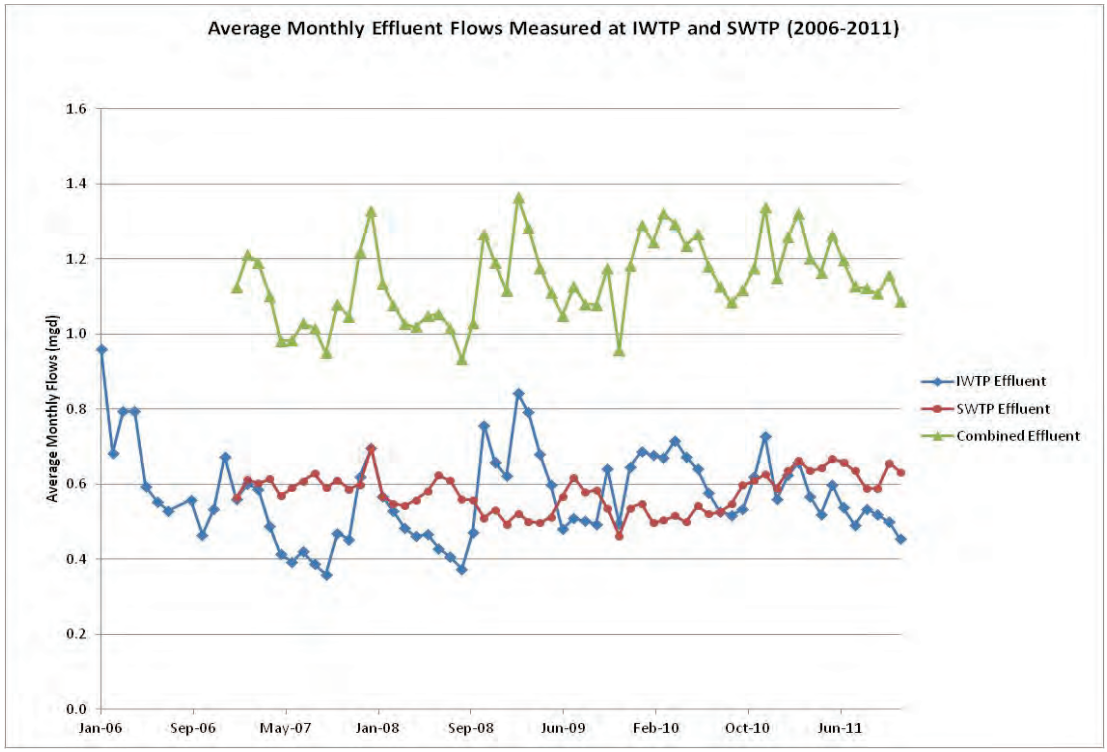
The annual average daily treated flow is the same (0.58 mgd) for the IWTP and the SWTP. The flow ratios in Table 2 indicate that the treated flows at SWTP show relatively small fluctuations which make blending the two streams an attractive option for increasing recycled water production capacity. However, water quality parameters need to be evaluated before considering the combination of the effluents from the IWTP and the SWTP. The average monthly flow from the IWTP, the SWTP and their combined values are presented in Exhibit 1.

Exhibit 1 Average Monthly Flows Measured at IWTP, SWTP and Their Combined Values (2006-2010)



Evaluation of Water Quality Data

Water quality has a significant impact on the technology selection, the size of the treatment plant unit processes, the need for pretreatment, chemical requirement, sludge and solids generation, capital and operations and maintenance (O&M) costs of the facilities.

The available daily historic water quality data for the IWTP included the following key parameters: influent and effluent total suspended solids (TSS); effluent turbidity; influent and effluent conductivity; influent and effluent biochemical oxygen demand (BOD), available three days per week; and dissolved oxygen (DO), available after October 2007. The analysis presented herein is based on 5 years of data between January 2006 and December 2010.

During the evaluation period, the average biochemical oxygen demand (BOD), total suspended solids (TSS) and ammonia as nitrogen (ammonia-N) values at the IWTP influent were approximately 17, 49 and 2 mg/L; respectively, This indicates a low strength wastewater (i.e., BOD <100 mg/L, TSS<100 mg/L). The influent pH values were typically between 7 and 8.5 which are very suitable for biological treatment.. The water quality parameters for the IWTP influent and effluent are summarized in Appendix A.

IWTP Effluent Quality and Plant Performance

The IWTP is required to meet the existing waste discharge Order No. R2-2007-0060 and NPDES No. CA0028070 issued by California Regional Water Quality Control Board for San Francisco Bay Region. The existing NPDES permit is valid until September 30, 2012.

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Because the provided water quality data does not cover upstream and downstream information for each unit treatment process, the performance of individual unit processes could not be determined with the existing information. Evaluating performance of the individual unit processes requires composite sampling and water quality analysis (minimum of one year) form upstream and downstream of the unit processes to determine removal efficiencies of key water quality parameters (i.e., BOD, TSS, etc.).

During the evaluation period (2006 -2010), the average effluent BOD concentration was relatively low (2.4 mg/L) which yielded an overall average BOD removal efficiency of approximately 86% at the IWTP as presented in Table 3.. In an average monthly basis, the IWTP has consistently met its maximum monthly discharge limit of 30 mg/L for BOD during the data evaluation period (Exhibit 2). It should be noted that in August 2006, effluent BOD values were significantly high for 5 consecutive days and were reported as high as 138 mg/L. Based on discussions with the SFIA project team, SFIA determined that this was a one-time aberration and CH2M HILL should exclude it from the analysis. The outlier data has been excluded for the data analysis as shown in Table 3.

Table 3 IWTP BOD, Conductivity and DO Values

		BOD			Conductivity		DO
		Influent	Effluent	Percent Removal	Influent	Effluent	Effluent
		mg/L	mg/L	%	umhos/cm	umhos/cm	mg/L
2006	**Ave**	16.5	2.3	84%	1829	1894	-
	Min	5.8	1.0	41%	592	652	-
	Max	51.6	7.1	96%	9647	8686	-
	Stdev	8.2	1.1	8%	950	954	-
2007	**Ave**	20.2	2.8	85%	1288	1338	7.4
	Min	7.2	1.7	61%	576	671	6.1
	Max	42.7	8.2	93%	2180	2069	9.1
	Stdev	7.0	1.1	6%	334	326	0.6
2008	**Ave**	20.4	2.6	86%	1766	1760	7.2
	Min	4.3	2.0	53%	617	692	5.0
	Max	138.0	10.0	97%	3128	2957	9.0
	Stdev	11.9	1.3	6%	521	491	0.8
2009	**Ave**	15.0	2.1	84%	1663	1630	7.2
	Min	5.5	2.0	64%	517	587	5.1
	Max	44.7	5.8	96%	6455	5948	9.2
	Stdev	5.8	0.6	5%	740	634	0.7
2010	**Ave**	11.1	2.1	79%	1162	1167	7.1
	Min	4.3	2.0	46%	523	590	5.1
	Max	22.9	5.1	91%	2240	2041	8.8
	Stdev	3.8	0.4	8%	314	269	0.7
Overall	**Ave**	**16.6**	**2.4**	**84%**	**1535**	**1554**	**7.2**
	Min	**4.3**	**1.0**	**41%**	**517**	**587**	**5.0**
	Max	**138**	**10**	**97%**	**9647**	**8686**	**9.2**
	Stdev	**8.5**	**1.0**	**7%**	**668**	**639**	**0.7**

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Exhibit 2 Effluent BOD Concentration at IWTP (2006-2010)

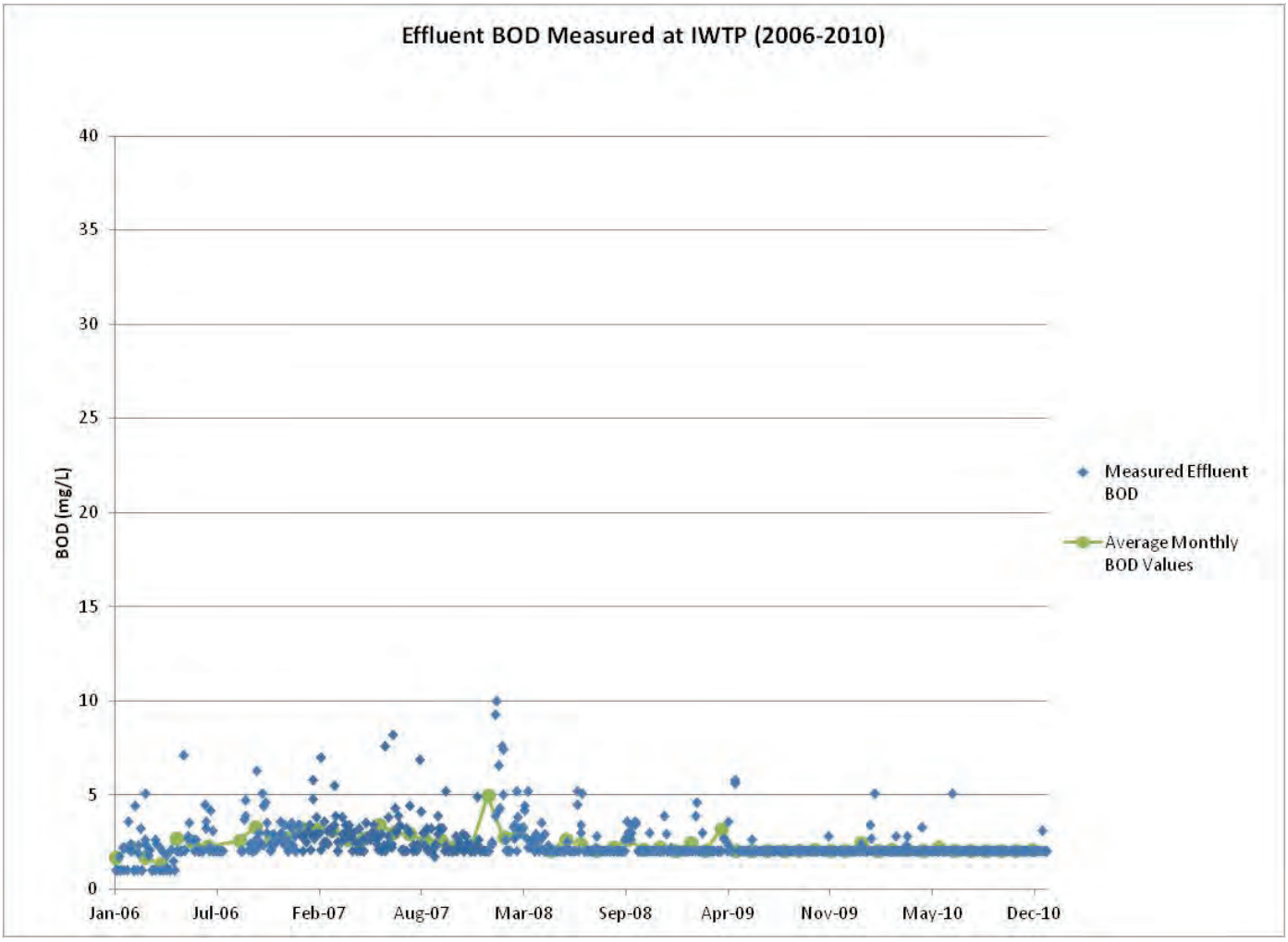

Table 4 summarizes the IWTP influent and effluent TSS concentrations and overall TSS removal efficiency of the plant. Effluent turbidity values measured at the IWTP effluent were also presented in Table 4. Exhibit 3 illustrates the effluent TSS measured at the IWTP. The average TSS concentration at the plant effluent was estimated at 4.3 mg/L; the data included a few spikes reaching 20 mg/L, which are comfortably less than the monthly average TSS limit of 30 mg/L in the NPDES permit. During the same evaluation period, the average TSS removal efficiency was approximately 90% which indicates a good solids removal performance at the IWTP. Effluent turbidity values and their implications are discussed in the next section.

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Table 4 IWTP Effluent TSS and Turbidity Values and TSS Removal Performance

		TSS			Turbidity				
		Influent	Effluent	Percent Removal	Effluent	No. of Times Turbidity Values > 5 NTU	Percentage of Time Turbidity Values > 5 NTU	No. of Times Turbidity Values > 10 NTU	Percentage of Time Turbidity Values > 10 NTU
		mg/L	mg/L	%	NTU	No.	%	No.	%
2006	**Ave**	40	3.9	89%	1.43	1	0%	0	0%
	Min	8	0.8	50%	0.36	-	-	-	-
	Max	129	14.0	98%	5.11	-	-	-	-
	Stdev	23	2.4	7%	0.84	-	-	-	-
2007	**Ave**	39	3.2	91%	1.08	3	1%	0	0%
	Min	12	0.6	44%	0.35	-	-	-	-
	Max	139	9.8	99%	6.38	-	-	-	-
	Stdev	17	1.8	7%	0.66	-	-	-	-
2008	**Ave**	52	3.8	90%	1.47	1	0%	0	0%
	Min	8	1.0	46%	0.50	-	-	-	-
	Max	290	11.4	99%	5.77	-	-	-	-
	Stdev	37	1.7	8%	0.66	-	-	-	-
2009	**Ave**	58	4.5	90%	1.98	6	2%	1	0.3%
	Min	9	1.0	66%	0.63	-	-	-	-
	Max	216	16.0	99%	10.20	-	-	-	-
	Stdev	34	2.2	6%	1.12	-	-	-	-
2010	**Ave**	53	5.4	87%	1.96	13	4%	0	0%
	Min	12	1.3	48%	0.42	-	-	-	-
	Max	367	17.7	98%	9.38	-	-	-	-
	Stdev	44	2.8	9%	1.32	-	-	-	-
Overall	**Ave**	**49**	**4.2**	**89%**	**1.58**	**24**	**1%**	**1**	**0.1%**
	Min	**8**	**0.6**	**44%**	**0.35**	**-**	**-**	**-**	**-**
	Max	**367**	**17.7**	**99%**	**10.2**	**-**	**-**	**-**	**-**
	Stdev	**33**	**2.3**	**8%**	**1.02**	**-**	**-**	**-**	**-**

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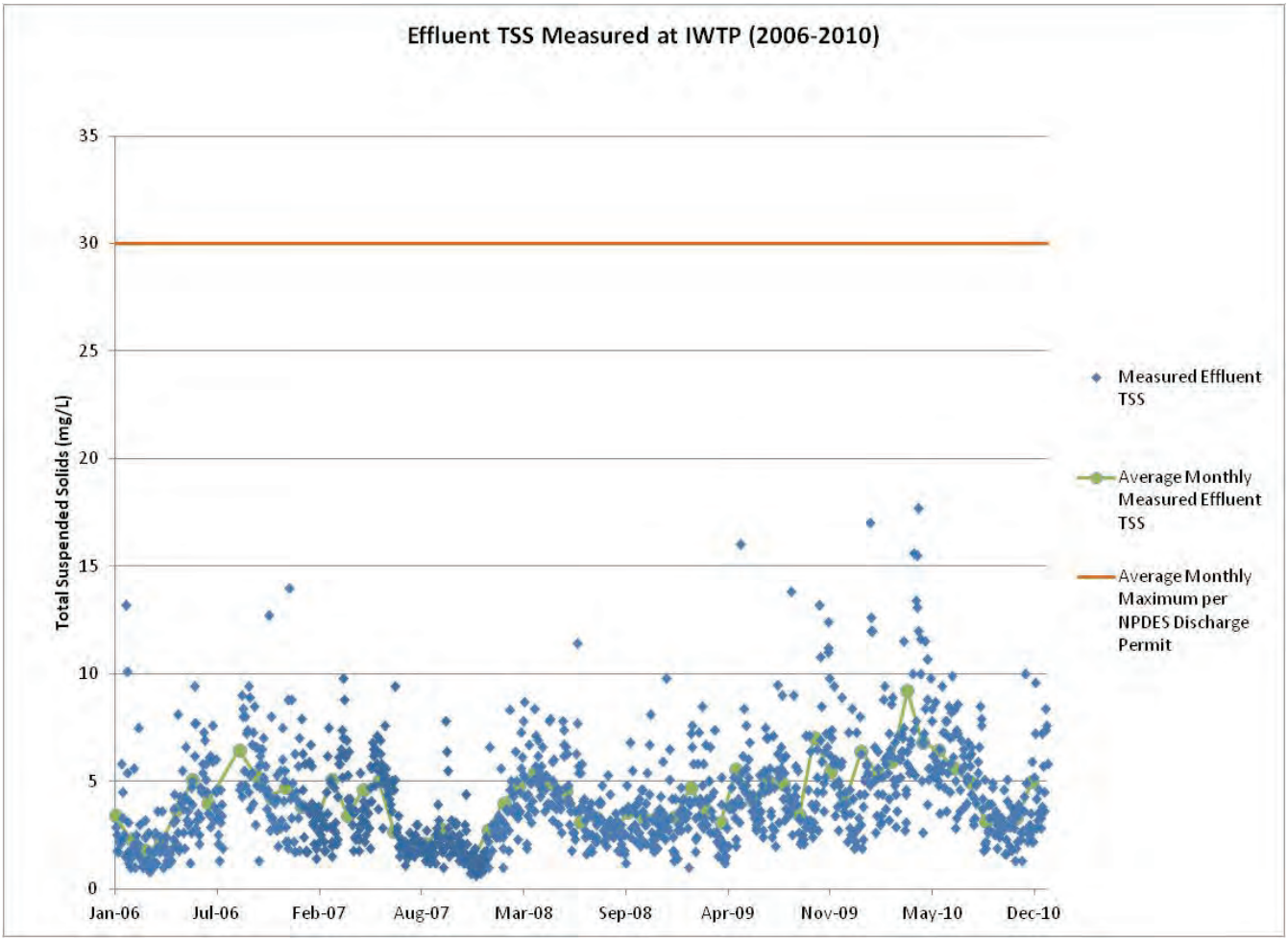
Exhibit 3 Effluent TSS Measured at IWTP (2006-2010)



Oil and grease concentration in the IWTP effluent included fluctuations during the evaluation period as summarized in Appendix A. In July 2007, a high value of 36.5 mg/L was reported in the plant effluent for oil and grease concentration. However, this high oil and grease concentration was due to a large spill that occurred in July 2007 based on information from SFIA project staff. The IWTP operation staff indicated that such large spills rarely occur and do not reflect typical oil and grease concentration in IWTP influent. In addition, the standard current practice at SFIA is to collect the spills in one of the containment basins and slowly feed it to the IWTP. Without this one-time event in July 2007, oil and grease concentration in the IWTP effluent are relatively low (averaged at 2.5 mg/L). High oil and grease concentration could pose problems in the treatment facility such as reduction in the oxygen transfer and mass transfer of substrate within the biofilm in trickling filters, and foam creation. In addition, oil and grease control is important for the performance of tertiary treatment facilities where oil and grease fouls filters and UV lamps. Therefore, implementation of effective oil and grease removal practices is essential for tertiary treatment performance.

Effluent Water Quality and Plant Performance Relevant to Tertiary Treatment

The IWTP will require tertiary treatment facilities (filtration and disinfection) to produce disinfected tertiary treated Title 22 recycled water. California Department of Public Health (CDPH) defines disinfected tertiary recycled water as "oxidized, filtered and subsequently disinfected wastewater" that meets the criteria presented in Table 5.

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Table 5 Water Quality Requirement for Disinfected Tertiary Treated Recycled Water

Parameter	Requirement
Organic material	Wastewater has to be properly oxidized under all conditions. " 'Oxidized wastewater' means wastewater in which the organic matter has been stabilized, is nonputrescible, and contains dissolved oxygen." All biological treatment unit processes shall be provided with reliability features such as "alarms and multiple biological treatment units capable of producing oxidized wastewater with one unit not in operation."
Turbidity (if treated through natural undisturbed soils or a bed of filter; i.e., dual or mixed media, upflow or pressure filtration systems, traveling bridge automatic filters)	Average of 2 NTU within 24-hour period 5 NTU not more than 5 percent of the time during 24-hour period Less than 10 NTU at all times
Turbidity (if treated through microfiltration, ultrafiltration, nanofiltration, or reverse osmosis [RO] membranes)	Average of 0.1 NTU within 24-hour period 0.2 NTU not more than 5 percent of the time during 24-hour period Less than 0.5 NTU at all times
Disinfection	A chlorine disinfection process following filtration that provides a CT (the product of total chlorine residual and modal CT measured at the same point) value of not less than 450 milligram-minutes per liter at all times with a modal CT of at least 90 minutes, based on peak dry weather design flow; or A disinfection process that, when combined with the filtration process, has been demonstrated to inactivate and/or remove 99.999 percent of the plaque forming units of F-specific bacteriophage MS2, or polio virus in the wastewater.
Total coliform bacteria	2.2 MPN per 100 mL per sample, median reading not to exceed over any 7-day continuous period 23 MPN per 100 mL per sample, not to occur more than once within 30 days 240 MPN per 100 mL in any sample

Recent communication with SFIA project staff revealed that the IWTP is expected to treat a very small flow during summer (especially if the United Airlines Facilities are relocated). The average year recycled water demand of 0.43-0.59 mgd cannot be satisfied by the IWTP alone and requires a more reliable source water. The SWTP has capacity and can provide the source water for tertiary treatment year round. The tertiary facility design will, therefore, be based on the SWTP effluent quality.

The dissolved oxygen (DO) concentration in the IWTP effluent during the evaluation period was between 5.0 and 9.2 mg/L with an average value of 7.2 mg/L thereby indicating that the wastewater was oxidized, a CDPH requirement for Title 22 unrestricted reuse application.

Because the IWTP secondary effluent quality is superior to SWTP effluent in terms of solids (i.e., TSS, turbidity) and BOD and ammonia-N, providing provision to treat flows from IWTP will be beneficial for tertiary treatment perspective. This provision will reduce operation cost of the tertiary treatment facilities. The proposed tertiary facilities will be designed with flexibility to treat secondary effluent from the following:

1. Only SWTP
2. Only IWTP (Seasonal)
3. Combination of SWTP and IWTP

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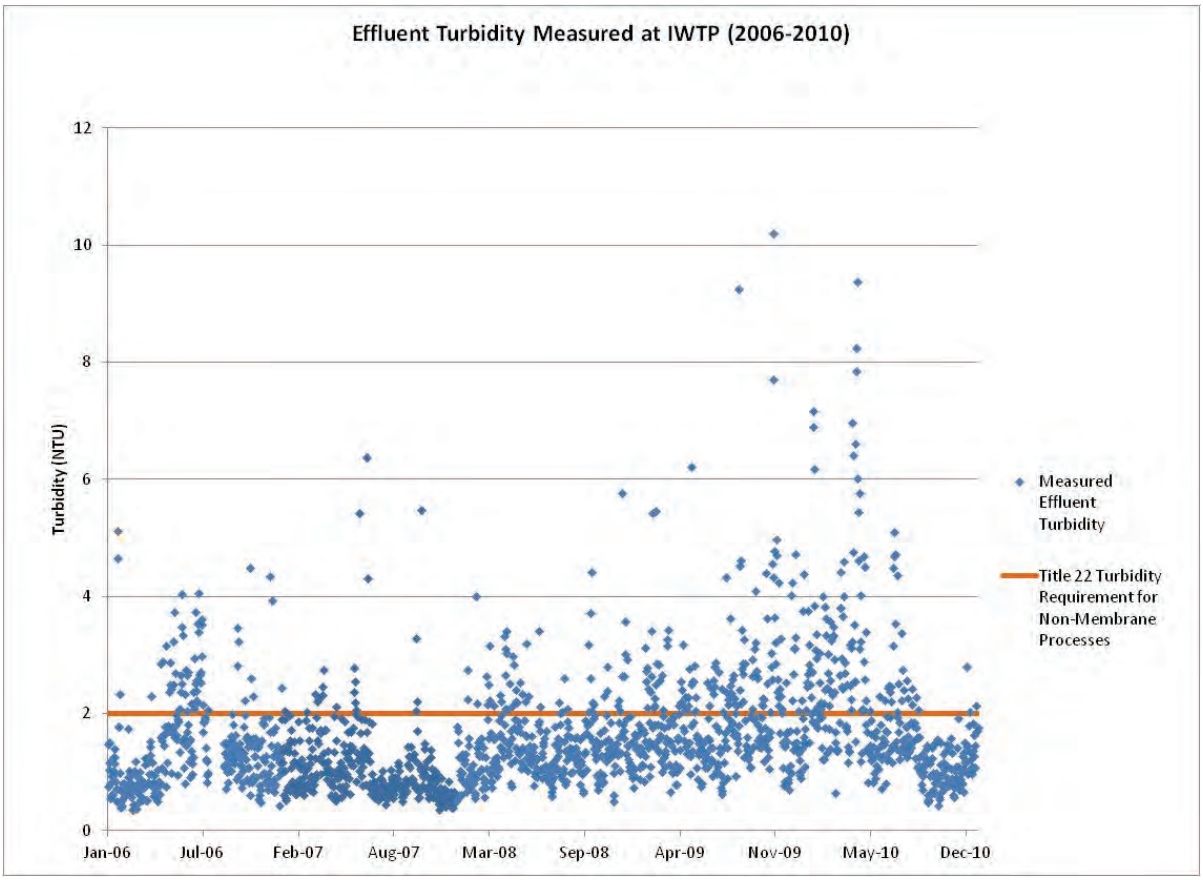
Feed water quality is of utmost importance for designing tertiary treatment facilities. Poor feed quality fouls filters, requires more frequent cleanings, reduces filter run time, generates larger volumes of backwash waste and increases chemical costs. In some cases, despite chemical use, certain non-membrane based filtration technologies (i.e., shallow bed traveling bridge filters) are not qualified by the CDPH to meet the turbidity requirement of 2 nephelometric turbidity units (NTU), if the turbidity in the influent to the filtration equipment exceeds 10 NTU

The secondary effluent (feed) water quality data from IWTP and SWTP that are relevant to tertiary treatment facility design are summarized below.

Exhibit 4 illustrates the effluent turbidity measurements relative to the Title 22 turbidity requirement for unrestricted use for non-membrane processes (i.e., conventional deep bed sand filtration, cloth disc media filtration, compressible medium filtration, etc.).

During the evaluation period, the IWTP has produced good effluent quality with an average effluent turbidity of 1.60 NTU which is less than the Title 22 turbidity requirement for non-membrane processes (2 NTU). During the 5 year data evaluation period, the effluent turbidity values exceeded 5 NTU thirty times and 10 NTU one time, respectively. In other words, the IWTP effluent has exceeded turbidity of 5 and 10 NTU about 2 percent and 0.1 percent of the time in 5 years. The IWTP effluent would be easy to treat with most filtration technologies to meet CDPH requirements for unrestricted use. Most CDPH approved filtration technologies are expected to meet the Title 22 turbidity requirements presented in Table 5.

Exhibit 4 Effluent Turbidity Measured at IWTP (2006-2010)



Using a conductivity to TDS ratio of between 1.3 and 1.7 and the average measured effluent conductivity of 1,580 µs/cm (Appendix A), the effluent TDS values at IWTP are estimated to be between 930 and 1,215 mg/L. During the same period, the maximum estimated TDS concentrations were between 5,100 and 6,880 µs/cm depending on the

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ratio used. Those high values, however, occurred less than 1% of the time for the 5-year data evaluation period. Title 22 recycled water regulations do not have a specific limit for TDS. The USEPA Reuse Guidelines recommend that the recycled water TDS be no higher than 2,000 mg/L. In general, the TDS content of IWTP secondary effluent is suitable for all purpose irrigation without further treatment. It is recommended that the tertiary treatment plant be furnished with an on-line conductivity meter to give flexibility to select low TDS source water (either IWTP or SWTP) for tertiary treatment.

It is also recommended that SFIA establishes the conductivity-TDS relationship at the IWTP and SWTP to better assess TDS content of water sources and the need for further treatment

Since a portion of recycled water may be used for industrial purpose (coiling tower), specific requirements for TDS and other water quality parameters should be discussed with the industrial user(s). CH2MHILL has requested specific water quality requirements (if any) from the industrial user(s).

Although not regulated by CDPH for non-potable reuse applications, both IWTP and SWTP effluent as well as the combined treated effluent contain substantial amount of ammonia (average of 1.7 mg/L for the IWTP and 62 mg/L for the SWTP). Certain cooling tower applications and industrial reuse schemes require no ammonia in the recycled water to protect their equipment and fixtures against ammonia induced corrosion. CH2MHILL has requested specific water quality requirements (if any) from the industrial user(s).

Although not regulated by CDPH, the presence of oil and grease in treated water is not desired. Oil and grease can foul membranes and filters used in tertiary filtration applications. Oil and grease can also foul UV lamp and exerts disinfectant demand which results in increased O&M costs. While the average oil and grease concentration is relatively low in the plant effluent (2.5 mg/L), a few oil and grease excursions (36.5 mg/L) have occurred during the data evaluation period due to large spills. It is recommended that the wastewater treatment upgrades consider treatment processes that are effective in removing oil and grease prior to tertiary treatment and the existing current practice to contain the spills and slowly feed it to the IWTP continues to be implemented.

The IWTP effluent contains metals, metalloids and fuel additives in trace quantities. These compounds cannot be removed during tertiary treatment unless coagulation/flocculation and other metal removal technologies are incorporated. Title 22 regulations for tertiary treated recycled water do not have specific limits for metals, metalloids and fuel additives. The USEPA Guidelines has recommendations for agricultural irrigation. However, these recommendations are expected to be more stringent for agricultural reuse than urban reuse due to direct human health as a result of edible crops consumption grown on recycled water. CH2MHILL has requested specific water quality requirements (if any) from the industrial user(s). These requirements also include metals and metalloids.

Sanitary Wastewater Treatment Plant Effluent Water Quality Data

Historic effluent water quality data for the SFIA Mel Leong SWTP was reviewed and analyzed, establishing average, minimum, maximum and standard deviation of key water quality parameter values. The analysis presented herein is based on historic water quality data provided by SFIA for SWTP effluent.

The daily historic water quality data available for the SWTP effluent included key parameters including: flows; TSS; turbidity; conductivity; carbonaceous biological oxygen demand (CBOD), available for three days per week; and DO (available after October 2007). The analysis presented herein is based on 5 years of data between January 2007 and December 2011. Yearly and overall average, minimum, maximum, and standard deviation of the historic data are presented in Table 6. A detailed summary of the historic data trends for these key parameters as well as several other parameters measured between January 2007 and December 2011 is provided in Appendix B. Effluent TDS concentrations were calculated for the SWTP assuming [TDS] = [Conductivity]/1.7.

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Table 6 Data Analysis of Key Water Quality Parameters Measured in the SWTP Effluent

		Flow		TSS	Turbidity	Conductivity	CBOD	DO
		Effluent	Combined Effluent	Effluent	Effluent	Effluent	Effluent	Effluent
		mgd	Mgd	mg/L	NTU	umhos/cm	mg/L	mg/L
2007	**Ave**	0.60	1.12	11	10	2230	11	7.4
	Min	0.44	0.71	6	4	1568	4	6.1
	Max	0.79	1.63	38	25	4449	24	9.1
	Stdev	0.06	0.18	3	4	343	4	0.6
2008	**Ave**	0.57	1.09	15	13	1900	12	7.2
	Min	0.38	0.74	7	7	1417	4	5.0
	Max	1.05	2.04	33	22	3514	27	9.0
	Stdev	0.09	0.17	5	3	239	4	0.8
2009	**Ave**	0.53	1.14	16	12	1725	11	7.2
	Min	0.34	0.74	8	5	1210	4	5.1
	Max	0.98	2.54	48	23	2599	24	9.2
	Stdev	0.08	0.19	5	4	215	4	0.7
2010	**Ave**	0.54	1.21	16	13	1586	11	7.1
	Min	0.38	0.84	7	4	1003	4	5.1
	Max	0.92	2.03	33	24	2516	27	8.8
	Stdev	0.08	0.16	5	5	309	5	0.7
2011	**Ave**	0.63	1.18	16	11	1651	10	6.9
	Min	0.39	0.72	7	5	1033	4	5.2
	Max	1.00	1.87	40	24	2494	24	8.8
	Stdev	0.09	0.15	4	4	261	4	0.5
Overall	**Ave**	**0.58**	**1.15**	**14**	**11**	**1820**	**11**	**7.1**
	Min	**0.34**	**0.71**	**6**	**4**	**1003**	**4**	**5.0**
	Max	**1.05**	**2.54**	**48**	**25**	**4449**	**27**	**9.2**
	Stdev	**0.09**	**0.17**	**5**	**4**	**361**	**4**	**0.7**

The secondary effluent TSS values ranged between 6 and 48 mg/L with an average of 14 mg/L. These SWTP TSS values are higher than the values observed at IWTP. The average turbidity value of 11 NTU with a value of as high as 25 NTU for the SWTP effluent indicates a relatively poor effluent quality for use as the feed for the filtration process (tertiary treatment). A turbidity exceeding 10 NTU in an average basis is a major concern because it:

- precludes the use of shallow bed filtration technologies (shallow bed filters are not approved by CDPH for filtration applications where feed water turbidity exceeds 10 NTU)
- requires pretreatment facilities (i.e., coagulation, flocculation and sedimentation) to meet Title 22 turbidity requirements (with the exception of membrane filtration)
- requires chemical use on a continuous basis with all filtration options (with the exception of membrane filtration
- makes filtration design and operation complicated and increases capital and O&M investment

The DO concentration in the SWTP effluent during the evaluation period was between 5.0 and 9.2 mg/L with an average value of 7.1 mg/L thereby indicating that the wastewater was oxidized and meets a CDPH requirement for Title 22 unrestricted reuse application.

Using a conductivity to TDS ratio of between 1.3 and 1.7 and the average effluent conductivity of 1,820 µs/cm (Table 6), the average TDS values at SWTP effluent are estimated to be between 1,070 and 1,400 mg/L. As mentioned previously, the USEPA Reuse Guidelines recommend that the recycled water TDS be no higher than 2,000 mg/L. In

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general, the TDS content of SWTP secondary effluent is suitable for all purpose irrigation without further treatment. However, in certain instances, the TDS concentration has exceeded 2,000 mg/L in two consecutive weeks. It is recommended that the tertiary treatment plant be furnished with an on-line conductivity meter to give flexibility to select low TDS source water (either IWTP or SWTP) for tertiary treatment.

It is also recommended that the San Francisco Airport to establish conductivity-TDS relationship at IWTP and SWTP to better assess TDS content of water sources and the need for further treatment.

Since a portion of recycled water may be used for industrial purpose (coiling towers), specific requirements for TDS and other water quality parameters should be discussed with the industrial user(s). CH2MHILL has requested specific water quality requirements (if any) from the industrial user(s).

Although not regulated by CDPH for non-potable reuse applications, SWTP effluent contains a substantial amount of ammonia (average of 62 mg/L and maximum daily concentration of 141 mg/L). The cooling tower application in San Francisco Airport, on the other hand, requires ammonia free recycled water. Therefore, ammonia in wastewater should be removed via biological or chemical methods or combination of both methods. The separate treatment methods as well as combined treatment options will be discussed in TM 3- Technology Selection for Treatment Alternatives.

Summary

The IWTP flow and water quality data between 2006 and 2010 were analyzed to characterize influent and secondary effluent quality. The flows and influent water quality will be the basis for designing preliminary, primary and secondary treatment facilities. The key flow and water quality data that will be used for the design of the preliminary, primary and secondary treatment facilities are presented in Table 7.

Table 7 Preliminary Key Design Parameters for Preliminary, Primary and Secondary Treatment Processes of the IWTP

Parameter	Value
Annual Average Day Flow, mgd	0.58
Maximum Monthly Flow (Qmax30), mgd	0.63
Minimum Day Flow, mgd	0.11
Maximum Day Flow, mgd	1.62
Average Influent BOD Concentration, mg/L	17
Maximum / Minimum Month Influent BOD Concentration, mg/L	35 / 181
Average Influent TSS Concentration, mg/L	50
Minimum / Maximum Month Influent TSS Concentration, mg/L	103 / 541
Average Influent Ammonia-N Concentration, mg/L	1.7
Minimum / Maximum Month Ammonia-N Concentration, mg/L	6.5 / 34
Average Influent pH	7.5
Minimum / Maximum Influent pH	6.6 / 9.6
Average Influent Temperature, oC	19.9
Minimum / Maximum Influent Temperature, oC	14.7 / 25.6

Although the IWTP secondary effluent quality is superior to that of SWTP, IWTP flows are seasonal and cannot satisfy recycled water demand year round. SWTP is the only source water that can reliably satisfy recycled water demand year round, consequently, SWTP will be the primary source for recycled water. The tertiary treatment facilities will be

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designed with flexibility to maximize the use of IWTP treated water, when available, to reduce O&M cost of the tertiary facilities. Tables 8 and 9 summarize key secondary effluent water quality data from SWTP and IWTP, respectively. These data will be used for the design of the tertiary treatment facilities.

Table 8 SWTP Key Secondary Effluent Flow and Water Quality Parameters For Tertiary Treatment

Parameter	Value
Annual Average Day Flow, mgd	0.58
Maximum Monthly Flow (Qmax30), mgd	0.70
Minimum / Maximum Day Flow, mgd	0.34 / 1.05
Average Feed (Secondary Effluent) BOD Concentration, mg/L	11
Maximum Feed (Secondary Effluent) BOD Concentration, mg/L	27
Average Feed TSS Concentration, mg/L	14
Maximum Feed TSS Concentration, mg/L	48
Average Feed Ammonia-N Concentration, mg/L	62
Maximum Day Feed Ammonia-N Concentration, mg/L	141
Average Feed Turbidity Concentration, NTU	11
Maximum Feed Turbidity Concentration, NTU	25
Percentage of Time the Feed Turbidity Exceed 5 NTU, %	85
Minimum Feed DO Concentration, mg/L	5
Average Feed Oil and Grease, mg/L	2.9
Maximum (Instantaneous) Feed Oil and Grease Concentration, mg/L	5.6

Table 9 IWTP Key Secondary Effluent Flow and Water Quality Parameters For Tertiary Treatment

Parameter	Value
Annual Average Day Flow, mgd	0.58
Maximum Monthly Flow (Qmax30), mgd	0.63
Minimum / Maximum Day Flow, mgd	0.11 / 1.62
Average Feed (Secondary Effluent) BOD Concentration, mg/L	<5
Average Feed TSS Concentration, mg/L	4.3
Maximum Feed TSS Concentration, mg/L	19
Average Feed Turbidity Concentration, NTU	<2
Maximum Feed Turbidity Concentration, NTU	10.2
Percentage of Time the Feed Turbidity Exceed 5 NTU, %	2
Minimum Feed DO Concentration, mg/L	5
Average Feed Oil and Grease, mg/L	2.5
Maximum (Instantaneous) Feed Oil and Grease Concentration, mg/L	36.5

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Appendix A
Industrial Wastewater Treatment Plant Data Analysis

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Appendix B
Sanitary Treatment Plant Data Analysis

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Appendix E.3
Technical Memorandum 3

FINAL TECHNICAL MEMORANDUM 3 CH2MHILL®

San Francisco International Airport Evaluation of Treatment Technology Alternatives

PREPARED FOR: San Francisco International Airport

PREPARED BY: CH2M HILL

DATE: April 2, 2012

Introduction and Objectives

The City and County of San Francisco owns and operates the Mel Leong Treatment Plant (MLTP) through the San Francisco International Airport (SFIA). The MLTP includes an Industrial Wastewater Treatment Plant (IWTP) and a Sanitary Wastewater Treatment Plant (SWTP). The IWTP treats industrial wastewater as well as first flush, storm water runoff from airport facilities and the SWTP treats the sanitary sewer from the airport facilities. The effluent from both facilities (IWTP and SWTP) is routed to either the effluent pump station or the water reclamation pump station. The combined effluent is pumped to the North Bayside System Unit outfall, where the treated water is combined with effluent from South San Francisco, Millbrae, and Burlingame. Dechlorination takes place in the shared outfall before the effluent is discharged in to the Bay. The effluent from the water reclamation pump station is used within the Airport for irrigation purposes under a restricted use permit. The sludge from both facilities is conveyed to sludge drying beds for dewatering and drying.

Technical Memorandum 2 (TM2) entitled "San Francisco International Airport Industrial Wastewater Treatment Plant Data Analysis" contains the analysis of the existing flow and water quality data to characterize the raw influent and treated effluent at the IWTP. This information will be used to establish the criteria and selecting treatment unit processes to meet SFIA's objective to produce effluent that meets the California Department of Public Health (CDPH) Title 22 requirements for unrestricted use.

The IWTP collection system collects wastewater from point sources from specific areas (maintenance shops, etc) and surface runoff. The point source wastewater is collected by the industrial wastewater collection system and pumped to the IWTP. The first part of runoff (first flush) is collected by first flush ponds. The collected runoff is then pumped to the IWTP at a controlled rate to avoid shock loading the IWTP. The existing IWTP is comprised of flow equalization, primary treatment (rapid mix and flocculation, and dissolved air flotation), secondary treatment (trickling filter and secondary clarifiers), and disinfection (using sodium hypochlorite). . The IWTP was built approximately X years ago and all its components are in need of replacement due to settlement and severe corrosion issues. The objective to produce Title 22 effluent for unrestricted use requires the new IWTP to include tertiary treatment in addition to the existing primary, secondary and disinfection processes at the existing facility.

This technical Memorandum 3 (TM 3) identifies and describes the different technologies available for each of the processes needed for the new IWTP. In addition, TM 3 contains the evaluation to short list two technologies for each type of treatment process. The short listed technologies will be assembled in up to two preliminary treatment train alternatives for the IWTP that will be described and evaluated in technical memorandum 4 (TM 4).

This TM contains the following sections:

1. Evaluation of Treatment Options
    - Primary Treatment
    - Secondary Treatment
    - Tertiary Treatment
    - Disinfection Treatment

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2. Short Listing of Treatment Technologies Using Multi Criteria Analysis
3. Summary of Technology Evaluation

1. Evaluation of Treatment Options

1.1 Primary Treatment Options

The wastewater flows vary and show significant daily and seasonal fluctuations. While the summer flows are projected to be approximately less than 0.1 million gallons per day (mgd), flows as high as 1.7 mgd have been recorded during wet periods. Therefore, the primary treatment should have the capability to treat peak flows and variable loads.

Primary treatment technologies typically focus on removal of settleable solids, suspended solids and oil and grease (O&G). In the case of the SFIA IWTP, primary treatment must also periodically handle oil/water emulsions in the influent.

The IWTP influent is not well characterized in terms of settleable solids and O&G content. However, influent total suspended solids (TSS) concentrations over the 2006-2010 time period ranged from a minimum of 8 milligrams per liter (mg/L) to as high as 367 mg/L with an average of about 49 mg/L. The amount of settleable solids such as dirt, sand and grit has not been measured; however, it is present in significant quantities as would be typically expected in industrial wastewater and first flush storm water. SFIA ITWP operators state that there is significant buildup of solids in the existing influent water equalization tanks. In addition, operating staff state that settled sludge builds up in the bottom of the existing dissolved air flotation (DAF) units such that they need to be periodically taken off line to clean out the settled solids sludge. This is because the existing DAF units do not have bottom sludge scrapers for solids removal.

Oil/water emulsions are also of concern to ITWP operators due to fuel spills that occur at the airport. Airport staff typically diverts any fuel spills into large existing storm water holding ponds for containment and equalization prior to sending the water to the IWTP for treatment. Despite this procedure, ITWP operators indicate that that oil/water emulsions can be problematic on occasion and should be addressed in the design of the new IWTP.

Primary treatment technologies employ physical means to remove settleable solids, suspended solids, O&G. Some of these technologies can treat oil/water emulsions as well. Treatment chemicals such as coagulants and flocculants are often used to enhance the performance of physical separation processes. Potentially applicable technologies for primary treatment for the IWTP include the following:

- Oil/Water Separators(OWS)
- Dissolved Air Flotation
- Primary Clarifiers

These technologies are briefly described and key issues and relative advantages and disadvantages are discussed.

a. Oil/Water Separator

Oil/water separators (OWS) are designed to separate free and dispersed oils and greases (as well as gross quantities of solids) from wastewater or process streams. Oil/water separators are usually the first stage of any wastewater treatment system. They operate on the principal of gravity differences between the water and the organic phase to separate one from the other. Oil/water separators can provide gross solids removal; however, the TSS in the effluent can still be relatively high.

There are a number of different types of OWS with some that include inclined corrugated plates to facilitate oil/water separation and to minimize equipment footprint size as indicate in Exhibit 1 below.

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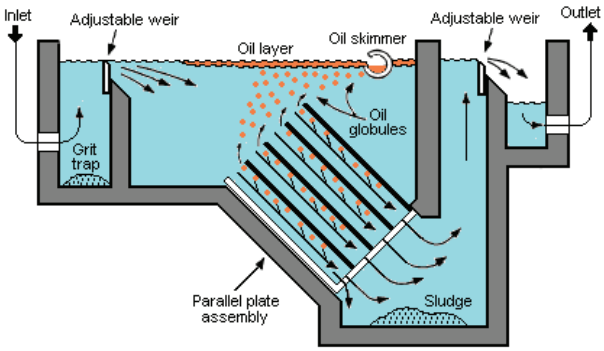
Exhibit 1 Oil /Water Separator Schematic



Advantages of Oil/Water Separators

- Simple to operate
- Can handle variations in flow rates
- Very efficient in gross oil phase removal and gross solids removal
- Proven technology
- Relatively small footprint compared to DAF and primary clarifiers

Disadvantages of Oil/Water Separators

- Cannot handle emulsions
- Cannot reduce TSS to low levels in effluent compared to DAF or primary clarifier
- Not a stand-alone technology in this application

b. Dissolved Air Flotation

The DAF process uses the principal of flotation to clarify wastewater by the removal of suspended matter such as oil or solids. The removal is achieved by dissolving air in the water or wastewater under pressure and then releasing the air at atmospheric pressure in a flotation tank or basin. The released air forms tiny bubbles which adhere to the suspended matter causing the suspended matter to float to the surface of the water where it may then be removed by a skimming device. Heavier settled solids can be removed by installation of sludge scrapers at the bottom. Coagulants and flocculants are commonly used in conjunction with DAF treatment. DAF treatment is currently being used successfully in the existing IWTP, except for the periodic need to remove accumulated sludge in the bottom of the DAF units. Exhibit 2 shows an example of a DAF unit.

Exhibit 2 Dissolved Air Flotation

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Advantages of DAF

- Can reduce O&G concentrations to lower levels compared to oil/water separator and primary clarification
- Effective in treating emulsions
- Very small or light particle that settle slowly can be removed more completely and in a shorter time; a high rate process compared to traditional gravity based settlement systems and typically requires less space and produces a thicker sludge;
- Sludge rakes can be installed at the bottom of a DAF unit to remove settled solids
- A stand-alone technology for O&G, gross solids, TSS and emulsion removal

Disadvantages of DAF

- Mechanically more complicated because of the need for air compressors/air handling systems
- Potential for higher air emissions and odor issues associated with air flow through the water
- Higher power load due to air compressors

c. Primary Clarifier

Primary clarification is a mainstay treatment technology commonly used for wastewater treatment based on separation of suspended particles by gravitational settling. This process is also effective for removal of O&G. Coagulants and flocculants are commonly used in conjunction with primary clarification. Clarification is currently being used successfully in the existing IWTP as part of downstream secondary treatment. Exhibit 3 shows an example of a primary clarifier.

Exhibit 3 Primary Clarifier

Primary Clarifier Advantages

- Simple proven process
- Excellent at TSS removal

Primary Clarifier Advantages

- Typically requires more space compared to DAF or oil/water separator
- Clarifier underflow sludge is much less dense compare to DAF sludge
- Not a stand-alone primary treatment technology because cannot handle gross solids and emulsions

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1.2 Secondary Treatment Options

The IWTP raw wastewater is a low strength wastewater with a relatively low average biological oxygen demand (BOD) and TSS concentration (each below 50 mg/L). The wastewater flows vary and show significant daily and seasonal fluctuations. This flow variation is from less than 0.1 mgd in the summer to as high as 1.7 mgd during the wet periods. Therefore, the secondary treatment should have the capacity to treat peak flows and variable loads. In addition, the selected treatment alternative should meet project objectives in a cost effective manner. The project water quality objectives are two-fold:

1. The treated wastewater should fully satisfy National Pollutant Discharge Elimination System (NPDES) permit limit for BOD, TSS and other specified constituents
2. The treated wastewater should satisfy Title 22 tertiary recycled water criteria when coupled with filtration and disinfection. This is required based on San Francisco Airport staff's interest to minimize NPDES discharges while maximizing water reuse

To meet these objectives, the following technologies were identified:

- Trickling filters (TFs)
- Sequencing batch reactors (SBRs)
- Rotating biological contactor (RBCs)
- Biologically aerated filter (BAFs)

The existing trickling filter-secondary clarification at the IWTP produces effluent turbidity ranging between 0.5 and 10.2 nephelometric turbidity unit (NTU) while averaging at 1.6 NTU (more than 98% of the time secondary effluent turbidities were less than 5 NTU between 2006 and 2010). This is a relatively good water quality that can satisfy Title 22 recycled water turbidity requirements when coupled with an approved filtration technology by the CDPH. It is expected that all four identified secondary treatment technologies can produce similar effluent quality in terms of solids and turbidity. There are other secondary treatment options that can meet project objectives with higher capital and operations and maintenance (O&M) costs. In addition, these alternatives (i.e., conventional activated sludge systems, membrane bioreactors) will also have difficulty in handling varying flows and loads. Therefore, conventional activated sludge (CAS) and membrane bioreactor (MBR) systems were not included in this evaluation.

a. Trickling Filters

Trickling filters are attached growth systems, which use fixed biofilm carriers to treat wastewater. Primary effluent is either pumped or flows by gravity to a trickling filter distribution system and trickles down over the biofilm surface. Air moves upward or downward to provide oxygen to support biological growth. Biofilm develops on biofilm carriers. Trickling filter components typically include a distribution system, containment structure, rock or plastic biofilm carrier, underdrain and ventilation system. Trickling filters produce solids that need to be separated using a liquid-solids separation process (i.e., secondary clarification, dissolved air flotation). Therefore, a trickling filter process typically includes an influent pumping station, trickling filter(s), trickling filter recirculation pumping station, and a liquid-solids separation unit process. Exhibit 4 shows an example of a hydraulically driven rotary distributor.

Ideal trickling filter biofilm carriers, or media, provide a high specific surface area, low cost, high durability, and high enough porosity to avoid clogging and promote ventilation (Metcalf and Eddy, 2003). Trickling filter biofilm carriers include rock, random (synthetic), vertical flow (synthetic), and 60°crossflow (synthetic) media. Modular plastic trickling filter media (i.e., self-supporting vertical flow or crossflow modules) is used almost exclusively for new trickling-filter-based wastewater treatment facilities.

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Exhibit 4 Hydraulically Driven Rotary Distributor

Trickling filters are the most commonly utilized attached growth systems; and with proper design, they can produce treated effluent that would be conducive to Title 22 use with tertiary filtration followed with disinfection. Trickling filters are relatively simple to operate and offer some advantages over attached growth systems (CAS, MBR, SBR)

Advantages of Trickling Filters:

- Simple to operate
- Require less energy compared to suspended growth systems (CAS, SBR, MBR)
- Proven technology
- Operating experience at IWTP
- Rapid recovery from shock flows and loads as well as toxic and inhibitory substances

Disadvantages of Trickling Filters:

- As in other fixed film systems, mass transfer rate of oxygen limits the process performance
- Effluent quality is not consistent due to random sloughing
- More prone to washouts
- Poor media wetting may lead to dry media pockets, ineffective treatment zones and odors
- Periodic snails control is required

b. Sequencing Batch Reactors

The SBR is an activated sludge process that involves a fill-and-draw, complete-mix reactor in which both aeration and clarification occur in a single reactor. The major difference between SBR and conventional continuous-flow, activated sludge system is that the SBR tank carries out the functions of equalization aeration and sedimentation in a time sequence rather than in the conventional space sequence of continuous-flow systems. The sequential phases comprise a cycle with defined time intervals to achieve certain objectives. The typical phases of each cycle are identified below and presented in Exhibit 5.

- Fill (raw or settled wastewater fed to the reactor);
- React (aeration/mixing of the reactor contents);
- Settle (quiescent settling and separation of MLSS from the treated wastewater);

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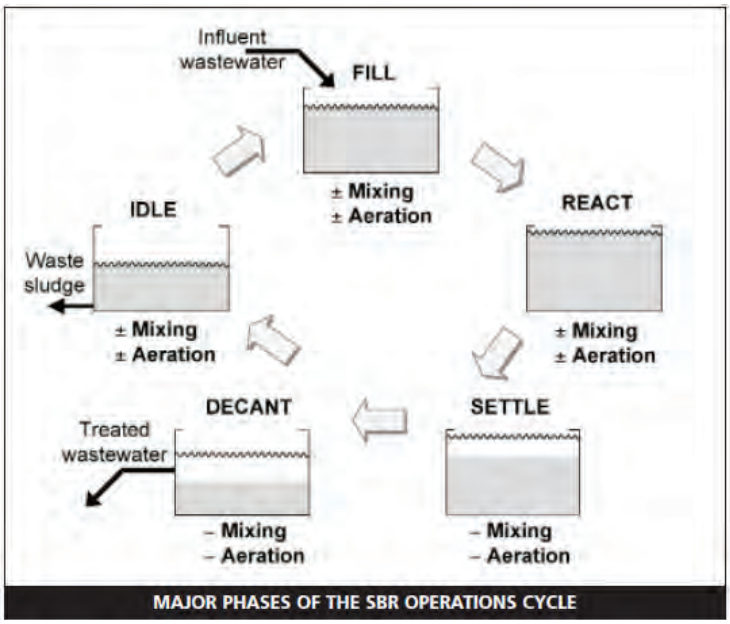
- Draw/decant (withdrawal of treated wastewater from the reactor); and
- Idle (delay period before beginning the next cycle and might include removal of waste sludge from the reactor bottom).

Exhibit 5 SBR Phases for Each Operating Cycle



The intermittent cycle extended aeration system (ICEAS) was developed in Australia as a modification to the typical SBR (Goronsky, 1979). Influent feeds continuously to the reactor during all cycles as in a continuous-flow system; but withdrawal is intermittent, similar to the SBR system (WEF, MOP 8, 2010).

Preliminary treatment is provided and it typically includes screening and grit removal. Primary settling is often not required. Elimination of secondary clarifiers will reduce foot-print of the facility. A picture of a 5-mgd SBR facility is presented in Exhibit 6.

Exhibit 6 A 5-mgd SBR Facility (Courtesy of Aqua Aerobic Systems Inc.)

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Advantages of SBRs:

- No need for secondary clarifiers and returned activated sludge (RAS) pumping systems.
- Clarification occurs under nearly ideal quiescent conditions which improves settling performance
- The fill, react, settle and idle times are adjustable that provide flexibility to meet project objectives

Disadvantages of SBRs:

- Completely mixed activated sludge (CMAS) reactors used in SBR systems can promote growth of filamentous bacteria that hinder sludge compaction. For larger plants, conventional plug flow (some with configuration flexibility) used in CAS systems is favored.
- Inability to effectively chlorinate RAS for filament control,
- Equalization of effluent decant may be required for subsequent downstream treatment
- Longer recovery period from shock flows and loadings

c. Rotating Biological Contactors

As a secondary treatment process, RBC has been applied where average effluent water-quality standards are less than or equal to 30-mg/L BOD and TSS. When the RBC is used in conjunction with effluent filtration, RBCs are capable of meeting more stringent effluent water-quality limits of 10-mg/L BOD and TSS. RBCs employ a cylindrical, synthetic media bundle that is mounted on a horizontal shaft. Exhibit 7 illustrates the shaft-mounted media. The media is partially submerged (typically 40%) and slowly (1 to 1.6 rpm) rotates to expose the biofilm to substrate in the bulk of the liquid (when submerged), and to air (when not submerged). Detached biofilm fragments suspended in the RBC effluent stream are removed by solids separation units. The RBC process typically is configured with stages operating in series. Each reactor-in-series may have one or more shafts. Parallel trains are implemented to provide additional surface area for biofilm development.

Exhibit 7 RBC cylindrical synthetic media bundle mounted on a horizontal shaft (left) and RBC covers (right) (Adapted from MOP 8, WEF, 2010)

Media-supporting shafts typically are rotated by mechanical-drives. Diffused air drive systems and an array of air-entraining cups that are fixed to the periphery of the media (to capture diffused air) have been used to rotate the shafts.

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Advantages of RBCs:

- Easy to operate
- Low operating cost
- Rapid recovery from shock loadings
- Cover tanks eliminate aerosol pollution

Disadvantages of RBCs:

- Susceptible to shaft, media or media support structure failures
- Macro fauna infestation
- Poor biofilm thickness control (results in inconsistent effluent water quality)
- Inadequate performance of air drive systems for shaft rotation
- Limited full-scale experience compared to trickling filters

d. Biologically Aerated Filters

Biological wastewater treatment and suspended-solids removal are carried out in BAF under either aerobic or anoxic conditions. In a BAF, the media acts simultaneously to support the growth of biomass and as a filtration medium to retain filtered solids. Accumulated solids are removed from the BAF through backwashing. There is a direct interaction between the media characteristics and the process, because the configuration (sunken media or floating media), and flow and backwash regimes depend on media density. Media may be natural mineral, structured plastic, or random plastic.

The BAF reactors can be used for carbon oxidation or BOD removal, only, combined BOD removal and nitrification, combined nitrification and dentrification, tertiary nitrification, and tertiary denitrification. The BAF reactors can be characterized into groups according to their media configurations and flow regime (WEF MOP 8):

Downflow BAF with media heavier than water: This general category includes both the Biocarbone® reactors commercially marketed in the 1980s for secondary and tertiary treatment and packed-bed tertiary denitrification reactors such as Tetra Denite® filters. These BAFs are backwashed using an intermittent countercurrent flow regime.

Upflow BAF with media heavier than water: This includes BAF reactors for secondary and tertiary treatment that use expanded clay and other mineral media, such as the Veolia Biofor®. These BAFs are backwashed using an intermittent concurrent flow regime.

BAF with floating media. This includes BAF with polystyrene, polypropylene, or polyethylene media, such as the Kruger Biostyr®. These BAFs are backwashed using an intermittent counter-current flow regime.

Continuous backwashing filters. These filters operate in an upflow mode and consist of media heavier than water that continuously moves downward, countercurrent to the wastewater flow. Media is directed continuously to a center air lift where it is scoured, rinsed, and returned to the top of the media bed.

With the exception of Down flow BAF with heavier media (i.e., Denite filters), the last three configurations can be used at IWTP. One commonly used BAF is the BIOFOR® BAF system supplied by Veolia. The BIOFOR® is a high-rate, up-flow biological fixed-film system. It employs a proprietary dense granular support media that acts as a biological contactor as well as a filter, thus eliminating the need for a separate clarification. The effluent to be treated enters continuously from the bottom of the reactor and is distributed over the entire filter surface area by the nozzle underdrain. The water then passes through the Biolite™ filter media which retain the suspended solids. Air is introduced continuously into the lower part of the reactor by Oxazur® air diffusers. The use of a co-current

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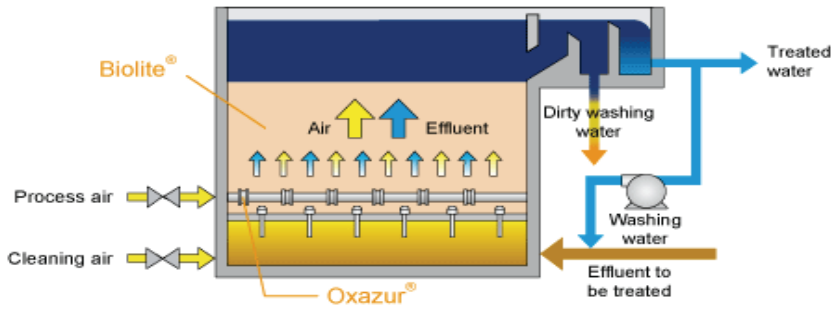
upflow design helps to limit odor generation since the treated water is situated at the surface of the filter (in contact with the atmosphere), and the untreated water enters at the bottom of the filter. Solids are trapped mostly in the lower part of the media bed during normal operation and are backwashed as required by increasing the hydraulic rate and applying scour air. As the backwash consists of concurrent scour air and backwash water, the accumulated solids travel up through the media bed before being released at the top. During low flow periods, off-duty filters are aerated periodically to maintain the biomass in optimum condition. The media is typically 10 to 13 feet deep with another 3 feet of freeboard. Exhibit 8 shows BIOFOR flow schematic.

Exhibit 8 Process Schematic of BIOFOR BAF (Courtesy of Veolia)



Advantages of BAFs

- Eliminates need for clarification or floatation for liquid/ solids separation which reduces foot-print and maintenance costs
- Better handle shock flows and loadings, less prone to washouts compared to SBRs
- Produces better effluent solids quality than all three identified candidates due to incorporating filtration

Disadvantages of BAFs

- As in other fixed film systems, mass transfer of oxygen limits the process performance
- Short circuiting and boils (violent eruption of the flow through the point of least resistance) occur when the filter bed or nozzles become blocked
- Produces backwash waste and requires backwash handling facilities
- Media loss may occur
- Requires skilled operation
- Limited full-scale experience compared to trickling filters

1.3 Tertiary Treatment Options

Tertiary treatment will include filtration and disinfection to produce Title 22 tertiary disinfected water criteria as presented in TM 2. The secondary treated effluent from the Sanitary Treatment Plant will serve as the feed (influent) water for the tertiary treatment facilities.

1.3.1 Filtration Options

Nearly two dozen tertiary filtration technologies have Title 22 approval for reuse applications. The filtration technologies used in reuse applications may be categorized as follows:

- Depth filtration
- Surface filtration
- Membrane filtration

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a. Depth Filtration

Depth filtration is one of the most common methods used for filtration of effluents from treatment processes, especially in water reuse applications. The types of depth filters used most commonly for wastewater filtration are:

- Conventional downflow filters (mono, dual, and multimedia)
- Deep bed downflow filters (mono, dual, and multimedia)
- Deep bed up-flow continuous backwash filters (e.g.; DynaSand)
- Compressible media filters (fuzzy filters)
- Shallow bed traveling bridge filters
- Shallow pulsed bed filters

Headloss buildup occurs as filtration takes place, and the system is backwashed one filter cell at a time. Filters that must be taken off-line periodically for backwash are classified as semi-continuous filters, whereas filters in which backwash and filtration operation occurs simultaneously are classified as continuous filters. Filter influent is conditioned using chemicals such as alum and polymer.

1. Conventional Downflow Filters

Flow containing suspended matter is applied to the top of the filter bed. Single, dual, and multimedia filter materials can be used. Sand and/or anthracite are the most common types used for reuse applications. Headloss buildup occurs as filtration takes place, and system must be backwashed routinely one filter cell at a time. They are classified as semi-continuous filters. They are approved by CDPH a filtration rate of up to 2 gallons per square foot (gpm/sq-ft). California example includes City of Stockton.

Advantages of Conventional Downflow Filters:

- Proven track record
- Can store solids within the filter bed which increases filter run length and reduces backwash waste
- Compatible with pretreatment chemicals
- Relatively easy to operate
-

Disadvantages of Conventional Downflow Filters:

- Typically higher capital costs
- Larger foot-print

2. Deep-bed Downflow Filters

The deep-bed filters are similar to conventional filters with the exception that the filter medium depth and the size of filtering medium are greater than those values in conventional filters. Because of greater depth and larger medium size, more solids can be stored within the filter bed and the filter run length can be extended. These filters are not generally fluidized completely during backwash thereby requiring air scour plus water for effective cleaning. They are also semi-continuous and approved for Title 22 applications by CDPH for a filtration rate of up to 2 gpm/sq-ft. The advantages and disadvantages of these filters are similar to the conventional downflow filters.

3. Deep-bed Upflow Continuous Backwash Filters

Deep bed (typically 40 inches or higher) upflow continuous backwash filters such as DynaSand, Siemens Astrasand, Westech Technasand, etc., require the chemically preconditioned wastewater to be introduced from the bottom of the filter where it flows upward through a series of riser tubes and is distributed evenly into the sand bed through the open bottom of an inlet distribution hood. The water flows upward through the downward-moving sand. Clean filtrate exits from the sand bed, overflows to a weir, and is discharged from the filter. Sand and trapped solids are drawn downward at the same elevation into the suction of an airlift pipe that is in the center of the filter. Compressed air is introduced to the bottom of the airlift to uplift sand and solids containing water. It is possible to

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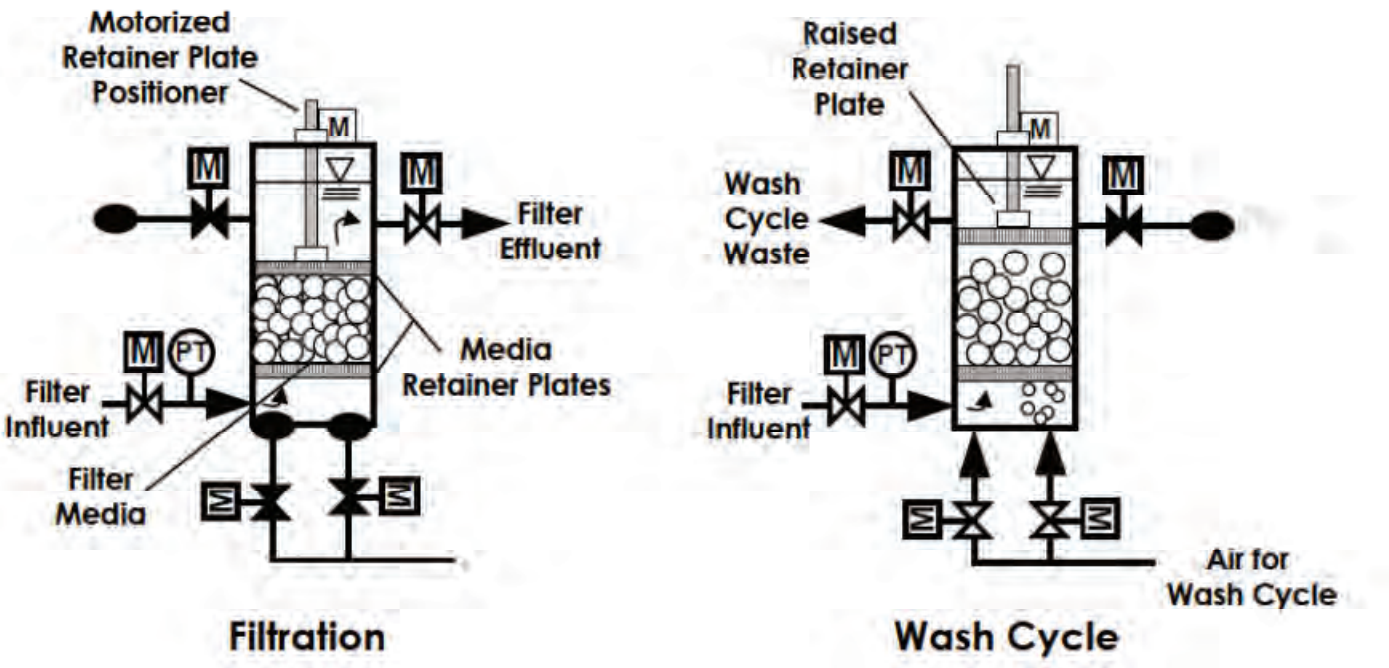
get sand blow off in the effluent, which can affect downstream disinfection. They are approved by CDPH a filtration rate of up to 2 gpm/sq-ft. Deep-bed up-flow continuous backwash filter users include Sausalito-Marin City Sanitary District and City of Rialto.

The major disadvantages of deep-bed upflow continuous backwash filters over the conventional and deep bed down flow filters are the media loss and increased backwash waste generation.

4. Compressible Medium Filters (Fuzzy Filters)

Compressible medium filter technology, a relatively new filtration technology, has unique properties. It involves use of a synthetic compressible fiber porous material as the filtering medium instead of conventional granular material (Caliskaner and Tchobanoglous, 2006). The media properties can be changed, by varying media compression, to meet the filtration requirements of different feed characteristics. Therefore, the filtration rates can be much higher than for conventional filters. A general schematic of the fuzzy filter system during both the filtration and wash cycles is shown in Exhibit 9.

Exhibit 9 Fuzzy Filter Operation Schematic (Adapted from Caliskaner et al. 2006)



During filtration, feed enters the bottom of the filter and travels upward through the media bed. The media consists of porous, compressible, synthetic-fiber balls that are 30 millimeters (mm; 1.25 inches) in diameter. Media depth is typically 30 inches but higher depths can be selected depending on applications. The media bed is supported on the bottom by a fixed retainer plate located above an influent plenum. On the top of the filter, the media bed is contained by a movable retainer plate. Both retainer plates have uniformly distributed holes to promote equal distribution of flow. The position of the top retainer plate can be changed by a motor and threaded shaft mounted to the top of the filter. By adjusting the position of the upper plate, the bed compression can be varied, depending on performance requirements.

CDPH approved the fuzzy filters for Title 22 applications at filtration rates of up to 30 gpm/sq-ft and at 30 inches uncompressed media depth and media effective size of 1.25 inches and uniformity coefficient of 1.50. There are few installations of fuzzy filters across the US, and only one installation in California (City of Yountville) as reported by CDPH.

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Advantages of Fuzzy Filters:

- Compact footprint due to higher filtration rates
- Media depth and porosity are adjustable to meet stringent water quality requirements

Disadvantages of Fuzzy Filters:

- Relatively new (no established track record on filter performance, chemical usage, backwash generation)
- O&M costs are not established
- Requires skilled operators

5. Shallow Bed Filters

Shallow bed filters have two main categories including traveling bridge automatic backwash filters (ABF) and shallow bed pulsed filters.

Traveling bridge ABFs are shallow bed (11 inches) continuous downflow, automatic backwash, low-head, and granular medium-depth filters. The filter bed is divided horizontally into long independent cells that treat the wastewater as it flows through them by gravity. A traveling bridge assembly is used to backwash each cell individually while other cells remain in service. Water used for backwashing is pumped directly from a clearwell plenum up through the medium and deposited in a backwash trough. Since backwashing is performed on an as-needed basis, the backwash cycle is termed semi-continuous. CDPH approval limits loading rate limited to 2 gpm/ sq-ft and maximum influent turbidity of 10 NTU. Examples of users of these applications include Sacramento County, Sepulveda Water Reclamation, Folsom WWTP, and Inland Empire Utility Agency Carbon Canyon Water Recycling Plant.

Shallow Bed Pulsed Filters are the derivatives of sand filters that use a shallow bed (10-12 inches). They are operated with a pulsing bed where pulses occur at least every 6 minutes, and no more than 25 pulses per filter run are maintained in Title 22 applications. CDPH approval limits loading rate limited to 2 gpm/ sq-ft and maximum influent turbidity of 10 NTU Title 22 applications include South Orange County Wastewater Authority, San Luis Obispo, San Clemente, Rancho Murrieta, Fallbrook, and others.

Advantages of Shallow Bed Filters:

- Low energy requirement due to low head losses
- Well established operating experience
- Typically generates less backwash waste than deep bed media filtration options

Disadvantages of Shallow Bed Filters:

- They are not approved for high feed turbidity applications (>10 NTU). They cannot be used at IWTP without full pretreatment (coagulation, flocculation, settling) to reduce feed turbidity less than 10 NTU at any given time
- Relatively large foot-print due to low filtration rates (2 gpm/sq-ft)

b. Surface Filtration

Surface filtration involves the removal of suspended materials by mechanical sieving by passing the liquid through a thin septum. Filter materials include cloth fabrics, woven metal fabrics, and variety of synthetic materials. The two common types of systems used in water reuse applications with Title 22 approval are the cloth media, cloth media

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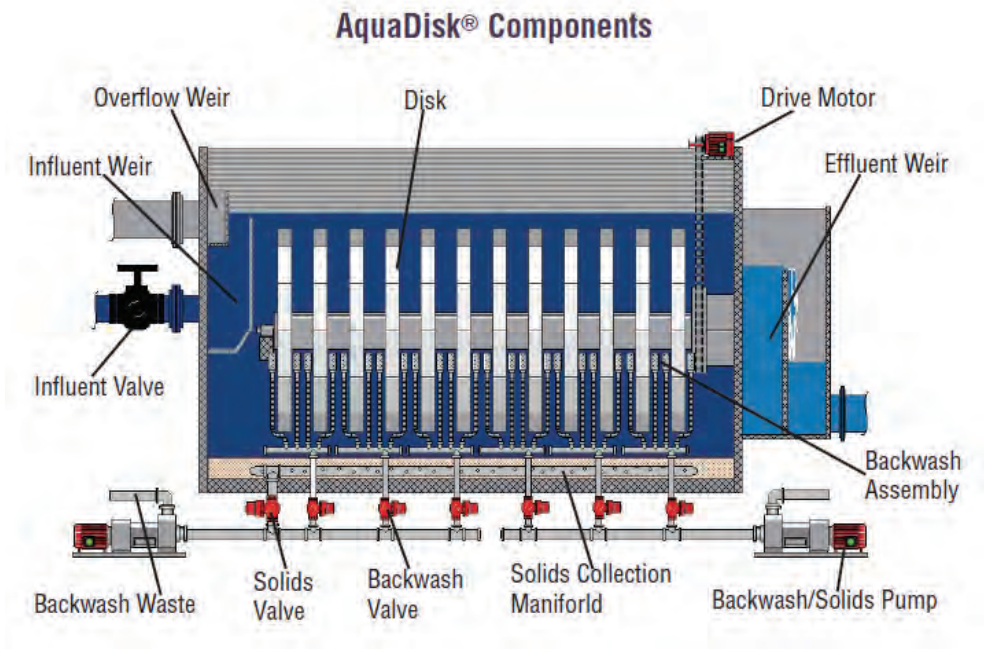
disc filters, and non-cloth disc filters (i.e., stainless steel media). These are relatively newer technologies, but there are a number of installations currently in operation.

1. Cloth Media Disc Filters (CMDFs)

The cloth media filters utilize nylon, acrylic, and polyester pile fabric to remove suspended materials from the water. The system is typically arranged as vertical disks (i.e., AquaDisk from Aqua Aerobic, Hydrotech from Kruger, Forty X from Siemens, etc.) or the fixed cloth media (AquaDiamond from Aqua Aerobic) in concrete or fabricated steel or stainless steel tanks. CMDFs are commonly used cloth filters that are available from multiple suppliers. They can be designed based on outside-in or inside-out flow configuration.

The most commonly used CMDF in California Title 22 applications is the AquaDisk supplied by Aqua Aerobic. The AquaDisk is designed to backwash automatically based upon water differential while maintaining continuous filtration during backwash. Each disk is made up of six pie-shaped sections which are mounted vertically to a common center tube, which conveys filtered effluent from the tank. This vertical media orientation allows for a large amount of filter area in a very small footprint (up to 75 percent less than typical filters). The filter is completely submerged during filtration with the disks only rotating during the backwash process. The pressure loss across the membrane increases as more particles are accumulated and a mat is formed on the surface of the membrane. The backwash operation is started when the terminal headloss (usually 12 inches of water) or a certain run time is reached. Accumulated particles are removed from the surface of the cloth membrane by liquid suction applied to each side of the disk. The accumulated solids are vacuumed from the media as the disk rotates past the shoes. Filtration occurs through the remainder of the filter disk, resulting in continuous filtration operation. Filtered water is used for backwash, therefore a separate clean water tank is not required as part of the backwash system. Large particles that have settled in the filter basin are removed through sludge assembly piping connected to the backwash pumps. Typical backwash is less than 2 to 3 percent (although this can be significantly higher during high solids loading), with a typical recovery time of less than 3 minutes. Exhibit 10 shows AquaDisk CDMF components.

Exhibit 10 AquaDisk CDMF Components (Courtesy of Aqua-Aerobic Systems Inc.)



AquaDisk uses 102-needle felt fabric, nylon pile fabric, acrylic pile fabric, and woven polyester fabric. Media openings are typically 10 microns. The CDPH-approved maximum filtration rate is 6 gpm/sq-ft. Applications of AquaDisk in tertiary filtration can be found in the USA. Title 22 applications include City of LA Tillman WRP, San Jacinto Valley RWRF, Temecula Valley Regional WRF, and Santa Margarita Water District Chiquita WWTP.

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Advantages of Cloth Media Disc Filters:

- Established track record
- Easy to operate and maintain
- Continuous operation during backwash
- Very low energy requirement
- Low backwash waste generation (typically less than 3%)

Disadvantages of Cloth Media Disc Filters:

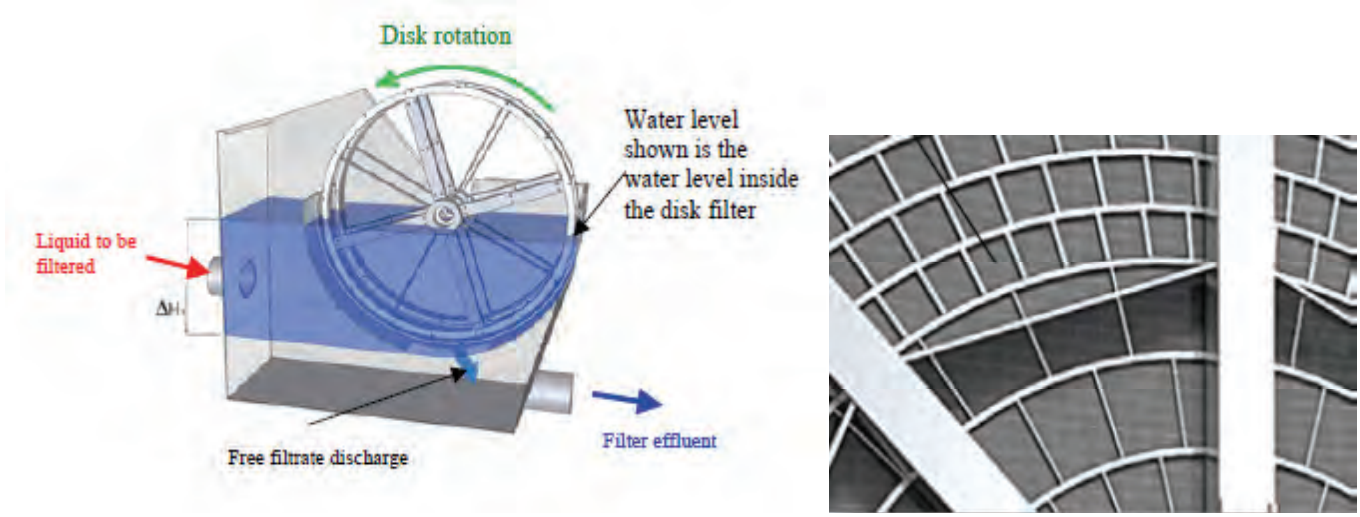
- High turbidity in Sanitary Wastewater Treatment Plant effluent requires chemical pretreatment. The pretreatment chemicals may adversely impact (media blinding occurs with polymer) CMDF performance
- High solids applications as in the IWTP may require sizing of the facilities at reduced filtration rates (i.e. <3 gpm/sq-ft) which increase capital cost and foot-print of the facilities

2. Non-Cloth Disc Filters

The Ultrascreen® is an inside-out surface filtration system that consists of continuously rotating disk filters made of AISI 316 stainless-steel screen mesh. The stainless-steel mesh screen (20-micron nominal-size) allows the Ultrascreen to handle higher hydrostatic heads, which translates to more efficient use of the total available filtering surface, potentially decreasing the overall footprint. The influent flows into the filter between each pair of disks, and then passes through the media. The filtered water freefalls into a collection well and exits through the outlet pipe. The wet edge of each disk is sealed to form a positive barrier to prevent the filtered effluent from mixing with the feed.

The disks are partially submerged in the water. Typically, 1 to 2 percent of the filter influent flow is being rejected. Exhibit 11 shows operating principles of Ultrascreen and media material.

Exhibit 11 Operating Principles of Ultrascreen® and Media Material



Demonstration studies have indicated that the Ultrascreen® can produce an effluent with a turbidity of less than 2 NTU while operating at a filtration rate of up to 16 gpm/ft^2, more than twice the hydraulic loading rate at which other commercially available disk filtration technologies operate (Burgeous et.al., 2009). However, the maximum filtration rate of 16 gpm/sq-ft is contingent upon it being complemented with a disinfection process that achieves 5-log inactivation/removal of the plaque-forming units of F-specific bacteriophage MS2, or polio virus. Therefore, CDPH approval, in most cases, is limited to a maximum filtration rate of 6 gpm/sq-ft, identical to those approved for all the other cloth disc media filters.

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Advantages of Non-Cloth Media Disc Filters:

- Easy to operate and maintain
- Continuous operation during backwash
- Very low energy requirement
- Low backwash waste generation (typically less than 3%)

Disadvantages of Non-Cloth Media Disc Filters:

- No full-scale facility in California for Title 22 applications
- Has larger nominal pore size (20 micron) than CDMFs (10-11 micron)
- High turbidity in Sanitary Wastewater Treatment Plant effluent requires chemical pretreatment. The pretreatment chemicals may adversely impact (media blinding occurs with polymer) CMDF performance
- High solids applications as in the IWTP may require sizing of the facilities at reduced filtration rates (i.e. <3 gpm/sq-ft) which increase capital cost and foot-print of the facilities

c. Membrane Filtration

Membrane filtration, micro filtration or ultra filtration (MF/UF), is a pressure-driven separation process that typically employs hollow-fiber membranes to block the passage of solids (i.e., turbidity, suspended solids) and pathogenic microorganisms, including bacteria and protozoa. While MF pore sizes range from about 0.1 – 0.2 micron (nominally 0.1 micron), UF pore sizes range from 0.01 – 0.05 micron (nominally 0.01 micron). Despite the pore-size differences, both systems produce virtually solids-free effluent with a turbidity typically less than 0.1 NTU without chemical addition for particle coagulation.

MF/UF can be configured as both pressurized (encased) or submerged systems. MF/UF systems are also classified based on flow pathway (outside-in vs. inside-out). For high solids feed water applications (i.e., reuse application), outside-in MF/UF systems generally perform better than inside-out configurations. In both types of systems, membrane fibers are bundled in groups of several thousand and potted in a resin on both ends to form a module, with tens to hundreds of modules coupled together to form a system. With pressurized type, the modules are housed in a pressure vessel or the vessel is integral to the module. Feed water is pressurized and applied to the feed side of the membranes in the module. Typical operating pressures range from 3 to 40 pounds per square inch (psi), depending on membrane technology and specific product operating conditions.

MF/UF systems are designed to filter small suspended solids and particles. Larger-size suspended solids, if allowed to enter the fiber bundle, can cause fiber damage (including breakage), and accumulate, leading to a buildup of solids. As a result, MF/UF systems employ self cleaning strainers with a screen size of 500 microns or less to prevent entry of larger particulates into the membrane modules. As constituents accumulate on the membranes, modules need to be backwashed or cleaned chemically to prevent membrane fouling. Exhibit 12 is a process schematic of a typical pressurized MF/UF system and example of an MF skid.

When coupled with a TDS removal technology (i.e., nanofiltration or reverse osmosis [RO]), only membrane filtration provides the adequate pretreatment and satisfies turbidity and silt density index (SDI) requirements for a sustainable RO operation (RO membrane manufacturers require feed turbidity and SDI values to be less than 1 NTU and 5, respectively while the goal is less than 0.2 NTU and 3, respectively, for sustainable RO system performance).

Other membrane material such as ceramic membranes and flat sheet and tubular configurations are able to meet Title 22 requirements. Because membranes are proprietary, the design filtration (flux), backwash, and chemical cleaning regimes may vary among suppliers. For small reuse applications (i.e., less than 5 mgd), pressurized hollow fiber membranes are the most cost-effective membrane technology (CH2MHILL, 2007).

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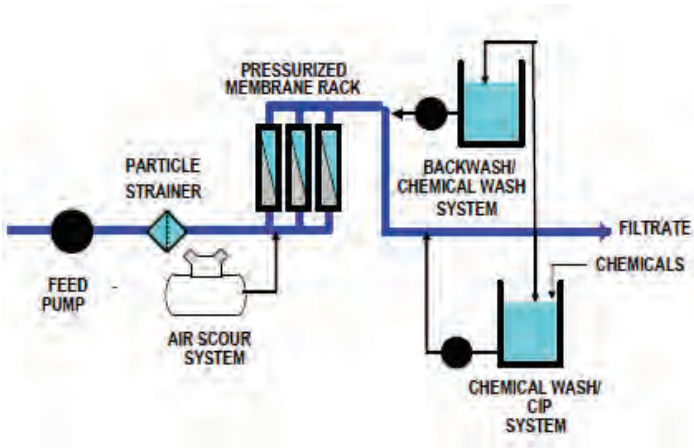
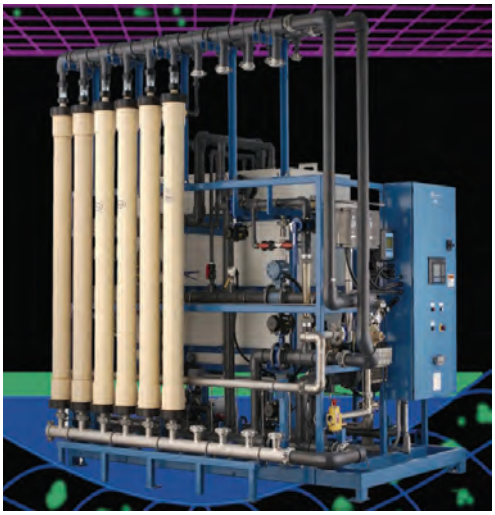
Exhibit 12 Process Schematic of a Typical Pressurized MF/UF System and Example of an MF Skid





California Title 22 MF/UF full-scale installation examples include Orange County Water District, Leo Vander Lans Advanced Water Treatment Facility, Lake Arrowhead and West Basin Municipal District.

Advantages of Membrane Filtration:

- Produce superior water quality with very low solids (i.e., turbidity less than 0.1 NTU)
- Meet water quality objectives independent of feed water quality and without requiring chemical pretreatment
- Proven technology
- Fully automated, relatively easy to operate
- Compact foot-print
- Reduces equipment requirement and O&M cost for the downstream UV disinfection facilities
- Reduces chlorine dose for chlorine disinfection

Disadvantages of Membrane Filtration:

- High capital and O&M costs
- Generates higher backwash volumes than many other filtration options (i.e., CDMF, shallow bed filters)
- Requires handling of chemicals for membrane cleaning

1.3.2 Disinfection Options

The objective of disinfection is to kill (inactivate) pathogenic microorganisms in the wastewater. Although chlorine dioxide is established for drinking water treatment, it is not used for wastewater, largely because only limited benefits are provided relative to chlorine disinfection of wastewater and they do not justify the additional system complexity and expense. Bromine species are proven disinfectants, but may also react with organic materials to form unwanted trihalomethanes (THMs). Brominated organics formed during disinfection are often considered more harmful than the analogous chlorinated organics (Water Environment Federation [WEF], 2006). Therefore, chlorine dioxide and bromine-based disinfection technologies are not included in this evaluation. Ferrate disinfection and peracetic acid could potentially meet Title 22 disinfection requirements. However, they are not cost effective and do not have CDPH approval for use in Title 22 applications, therefore, they were excluded from this evaluation.

The following CDPH approved disinfection technologies were evaluated:

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- Chlorine disinfection
- UV disinfection
- Ozonation
- Pasteurization

a. Chlorine Disinfection

Chlorine disinfection is the oldest and most commonly used disinfection technology in wastewater treatment and reuse applications. Chlorine in the form of gaseous chlorine or liquid sodium hypochlorite is injected into the wastewater. Chlorine disinfection should provide a minimum concentration contact-time (CT) requirement of 450 mg per minute per liter with a modal contact time of no less than 90 minutes enforced by the CDPH. Baffle walls are used to form a serpentine flow to provide adequate contact between disinfectant and wastewater pathogens. One of the major advantages of using chlorination is that the existing MLTP has liquid chlorine feed systems, which can be used with little or no upgrade.

Chlorination is highly effective for inactivating diverse pathogens including viruses. However, chlorination generates unwanted disinfection byproducts (DBPs), which have been strictly regulated in drinking water projects by the United States Environmental Protection Agency (USEPA) and local regulatory agencies. More recently, regulatory agencies and the public have raised concerns regarding DBPs in recycled water projects that necessitated consideration of alternative disinfection technologies such as UV disinfection, ozonation, etc., that result in lesser DBPs produced compared to chlorine-based disinfection.

Advantages of Chlorine Disinfection:

- Proven technology, excellent track record
- Water quality has a very little impact on disinfection performance
- Very effective on virus
- Easy to operate
- Familiarity of IWTP operators with the chlorine disinfection
- Low capital and O&M cost

Disadvantages of Chlorine Disinfection:

- Large foot-print
- Chemical to handle
- Generates disinfection by products

b. UV Disinfection

UV disinfection transfers electromagnetic energy from a mercury arc lamp to wastewater, emitting a broad spectrum of radiation. UV light penetrates an organism's cell walls and disrupts the cell's genetic material, making reproduction impossible.

With the proper dosage, UV irradiation has proven to be an effective disinfectant for bacteria, protozoa and viruses in recycled water, while not contributing to formation of DBPs. The principal electrode-type lamps used to produce UV light fall into the following three categories based on internal operating parameters:

- Low pressure-low intensity UV lamps
- Low pressure-high output UV lamps
- Medium pressure UV lamps

Low pressure lamps generate UV light with intense peaks at UV wavelengths of 253.7 nanometers (nm) and a lesser peak at 184.9 nm. Medium pressure lamps generate polychromatic light with peaks at multiple UV lengths, of which only approximately 7 to 15 percent is germicidal (200 to 300 nm, with 254 the optimum). The majority of the energy used by medium pressure lamps is wasted because their wavelength is outside the germicidal range. As a result,

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medium pressure system energy costs are higher than those of low-pressure systems. In addition to the type of UV lamps used, UV systems are often classified according to whether flow occurs in open or closed channels. For small- to mid-size water reuse applications, a closed-channel configuration using low pressure-high output lamps is the most commonly used and cost-effective technology. UV disinfection with its compact footprint is the disinfection technology finding wider use, especially following membrane technologies that produce high-quality effluent suitable for UV disinfection. This is especially true for expansions at treatment plants that are located within built-out settings.

There are multiple suppliers (i.e., Trojan, ITT Wedeco, Aquionics, Ultraguard, Degremont, Quay Technologies, etc.) that have received CDPH approval for Title 22 applications. The UV systems are designed based on National Water Research Institute (NWRI) UV Disinfection Guidelines for reuse. The UV reactor examples are presented in Exhibit 13.

Exhibit 13 UV Reactor Examples

Advantages of UV Disinfection:

- Small footprint
- Does not create disinfection by products
- Easy to operate
- No chemical to handle

Disadvantages of UV Disinfection:

- Water quality has a very little impact on disinfection performance
- High hardness can foul UV lamps
- Low UV transmittance, turbidity, nitrite and other UV scavengers reduce disinfection performance
- Sophisticated dose control required
- Not effective on virus inactivation
- Relatively high capital and O&M cost

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c. Ozonation

Ozone is a pale blue gas that forms when diatomic oxygen (O2) is excited to a higher energy state with a triatomic form (O3). This transformation occurs as oxygen is exposed to a source of high energy. Ozone can be generated onsite by passing a high-voltage alternating current (6-20 kilovolts [kV]) across a dielectric discharge gap through which oxygen or oxygen-laden air passes, resulting in the formation of ozone.

Ozone is used in water treatment plants for a variety of reasons, including primary disinfection, oxidation of taste and odor compounds, and control of chlorinated DBPs. Use at wastewater plants is less common, although it is gaining popularity because of its ability to simultaneously disinfect pathogens and oxidize trace organic compounds (e.g., chemicals of emerging concern). Ozone is a strong oxidant and disinfectant. Unlike free chlorine, however, ozone decays rapidly in aqueous solutions, and it does not maintain a residual concentration. Oxidation reactions are not selective; therefore, ozone reacts in water with a wide variety of materials (that is, both organic and inorganic materials). These reactions create an ozone demand that is characteristic to the specific water quality. Other DBPs may be formed from ozonation (bromate), but bromated formation depends on the bromide concentration in the water and can be minimized by pH adjustment and peroxide addition.

Ozone is a sparingly soluble gas in water, which usually requires longer CT to dissolve and become effective for disinfection. The traditional ozonation process requires large contact basins with multiple contact chambers. Ozone is bubbled in the first two or three chambers and subsequent chambers provide additional CT. CT requirements for disinfection are met when the effluent leaves the last chamber.

Pressurized plug-flow tubular reactors, where ozone can be added in sequential steps, allow ozone to dissolve faster in water and to utilize it more efficiently than using contact basins. HiPOX supplied by APT Water uses a tubular reactor (Exhibit 14) and is the only CDPH-approved reactor system for Title 22 applications.

The ozone doses are dependent on the feed water quality and typically between 2 and 5 mg/L in Title 22 applications. CT of as low as 5 mg/L-minute is sufficient to inactivate pathogens. Currently, no full-scale ozonation application exists in California for recycled water.

Exhibit 14 APT HiPOX Reactor

Advantages of Ozonation:

- Small footprint
- Does not create disinfection by products as much as chlorine

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- Strong oxidant; also remove emerging contaminants
- Performance is not very sensitive to water quality

Disadvantages of Ozonation:

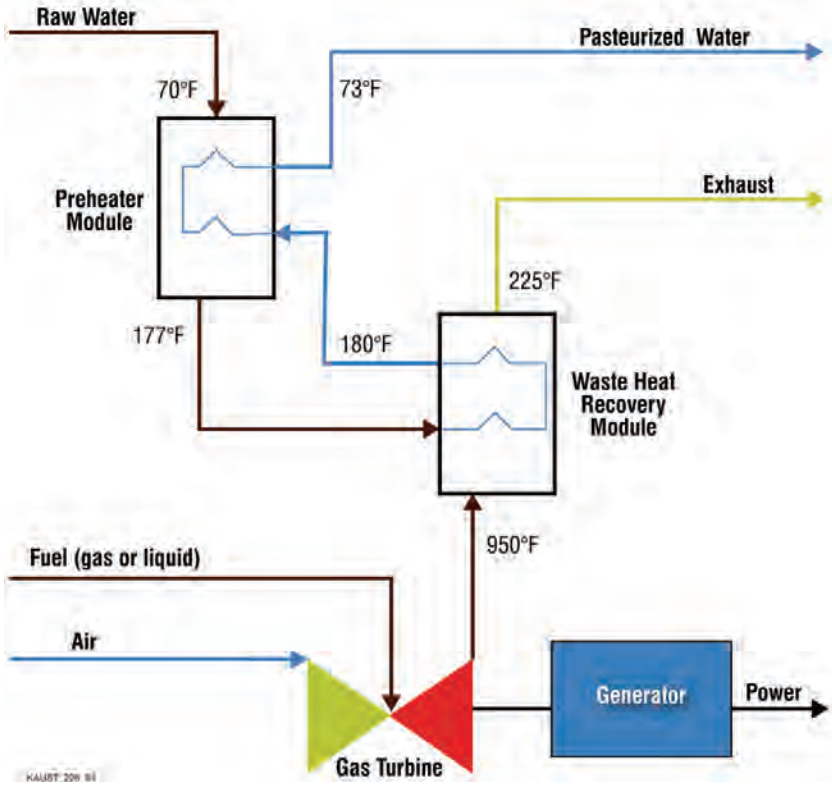
- High capital and O&M cost
- No full-scale application in wastewater and reuse applications
- Poor feed water quality (i.e, high turbidity, organics) increases ozone dose
- Sophisticated dose control required

d. Pasteurization

Pasteurization of water by boiling has long been practiced as a way of treating water contaminated with enteric pathogens. In fact, pasteurization can take place at much lower temperatures, depending on the length of time the water is held at the pasteurization temperature (Burch and Thomas, 1997). Pasteurization time decreases exponentially with increasing temperature. Above 50 degrees Celsius (°C), time decreases at roughly a factor of 10 for every 10°C increase in pasteurization temperature. Viruses appear the hardest to kill and essentially set the boundary for acceptable time–temperature processes. A typical process is 75°C for 10 minutes. The major advantage of pasteurization is that apparently all major pathogens of concern are killed, independent of wastewater quality; turbidity, pH, and other parameters that influence the efficacy of other disinfection methods.

Pasteurization has never been considered as a water/wastewater disinfection method because of the high energy cost in heating large volumes of water. Recently, Pasteurization Technology Group, USA, developed an innovative pasteurization concept where disinfection may be achieved at a reduced cost. The reduced cost of pasteurization is based upon the capture of a waste heat source (such as turbine exhaust, solar heat, or cooling towers) and the transfer of that heat to the water for disinfection, as depicted in Exhibit 15. The main components of this process are commercially available and include a gas engine generator and a set of heat exchangers.

Exhibit 15 Schematic of Pasteurization (Courtesy of Pasteurization Technology Group)



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There is no full-scale application of this technology in the US. Pilot studies were conducted at the Laguna Wastewater Treatment Plant (WWTP), in Santa Rosa for Title 22 approval. Conditional approval for reuse application was granted to the technology by CDPH on July 25, 2007. CDPH required that pasteurization temperatures of at least 82°C (180 degrees Farenheit [^{o}F]) be maintained continuously for a minimum of 10 seconds. This minimum temperature and CT must be demonstrated to the CDPH, spanning a range of flow from the lowest to the highest, with two intermediate flow points. Following successful demonstration and approval, the technology was pilot-tested in three municipal wastewater treatment plants (WWTPs) in California. One demonstration-scale (0.5-mgd) facility is under construction in California to further evaluate the technology and develop an economic analysis.

Potential Advantages of Pasteurization:

- Effective for inactivating virus, bacteria and other pathogens
- Does not create disinfection by products
- No chemical to handle
- Feed water quality has very little or no impact on disinfection performance

Disadvantages of Chlorine Pasteurization:

- New technology, no track record
- High capital and O&M costs
- Requires skilled operation
- Requires large heat exchanger to reduce water temperature on treated effluent
- Susceptible to calcium carbonate scaling

2. Short Listing Treatment Alternatives Using Multi Criteria Analysis

CH2M HILL's proprietary software, SMART, uses multi criteria analysis (MCA) methodology to develop clear and defensible benefit scores for each candidate treatment technology. With SMART, a set of criteria is developed for use in ranking the appropriateness of each alternative to satisfy project objectives. Then each criterion is assigned a weight factor that reflects its relative importance. The weighting factors range from one (least important relative to other criteria) to ten (most important relative to other criteria), allowing calculation of a weighted criterion score based on how important the criterion is for the project in the decision-making process for short listing the treatment technologies.

CH2M HILL developed candidate criteria then submitted these criteria to SFIA for review, comment, and then endorsement. Table 1 summarizes the evaluation criteria, criteria definition and respective weight resulting from this collaborative process for preliminary/primary, secondary and tertiary filtration options. The evaluation criteria for the disinfection options are the same as the information presented in Table, except the "Backwash waste / reject/ sludge generation" criterion was modified because it does not apply to the disinfection process. For the disinfection options, the criterion was revised to "DBP Generation and Compatibility with Future Regulations" which has the following description "The extent the technology generates DBPs and capability of technology to meet stringent water quality requirements in future without no or minor modifications/upgrade". The weighting factor for this criterion is 5.

Each criterion received a score from 0 to 5 where, 0 reflects fatal flaw (technology is not acceptable), 1 reflects the least favorable and 5 reflects the most favorable score for each criterion.

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Table 1 Non Monetary Criteria, Criteria Description and Weighting Factors for Preliminary/Primary, Secondary and Filtration Options

Criteria	Description	Weighting (importance) Factor
Reliable and consistent operation	Ability of the alternative to reliably treat wastewater regardless of influent wastewater quality conditions	10
Operating experience (Proven Technology)	The extent of successful full-scale operations	10
Ease of O&M	Relative degree of ease and extent of time required to operate and maintain the treatment facilities	10
Ease of expansion	Ability of the alternative to be expanded	4
Air quality impact	The extent the technology contributes to air pollution and degree of difficulty obtaining air quality permit	5
Ease of incorporating into the existing facility	Degree of design and construction necessary to integrate alternative into existing plant, difficulty/time required for such integration, and extent of impact on ongoing operations	6
Backwash waste/reject/sludge generation	The extent of backwash waste/reject generated from the facility	5
Power requirement/ carbon footprint	Extent of power consumption and carbon footprint of the technology	6
Olfactory impacts/overall aesthetics	Appearance of the facilities associated with the alternative and its visual/auditory/olfactory impact on project locations (height, noise, odor) and impacts of these effects on plant operators and surrounding neighborhood	2
Chemical usage	Relative number of chemical storage and feed facilities and hazardous nature of chemicals to be stored	1
Footprint	Space requirement of each technology	1

Multi Criteria Analysis Results

The following section discusses the results of the multi criteria analysis and presents a graphical summary of the results for each technology type. Appendix A includes the detailed numerical rating spreadsheets from which the graphical summaries were developed.

Primary Treatment Options

The SMART Multi criteria analysis results for primary treatment options showed that DAF is the most suitable option with the highest combined benefit scores of 4.02 (Exhibit 16). DAF is a well established and proven technology and is currently being used successfully at the IWTP.

On a practical level, DAF is a complete stand alone treatment process that can handle O&G, suspended solids and emulsions. This is a clear advantage over primary clarifier that cannot handle emulsions and over OWS that cannot handle fine suspended solids or emulsions. DAF treatment is recommended for primary treatment.

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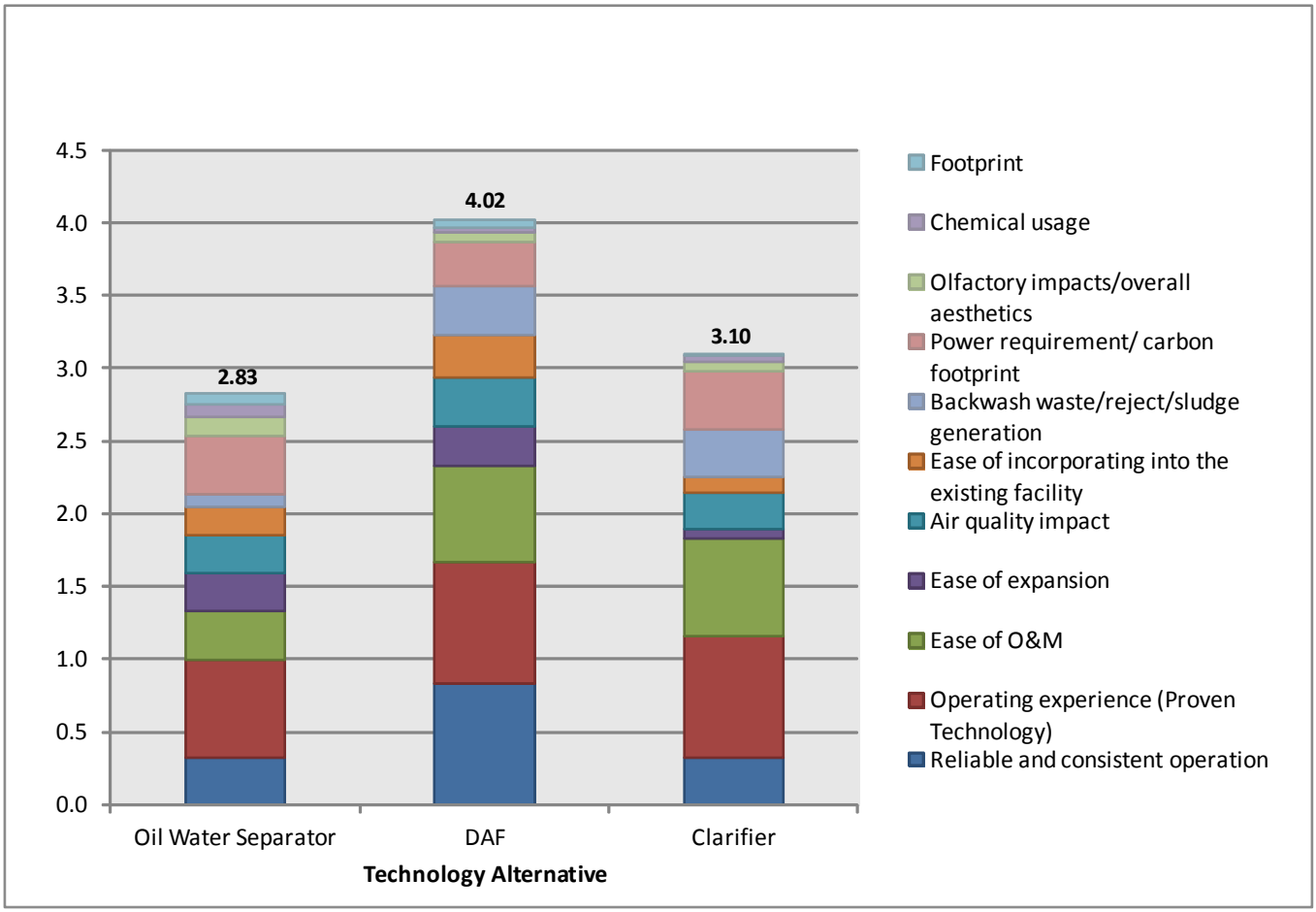
Exhibit 16 SMART Analysis Results for Primary treatment



Secondary Treatment Options

The SMART Multi criteria analysis results for secondary treatment options showed that trickling filter is the most suitable option with the highest combined benefit scores of 3.77 (Exhibit 17). Trickling filters have been a choice for treating wastewater flows and loads exhibiting significant fluctuations as in IWTP. SFIA is currently operating trickling filter at the IWTP which all together maximized the score for operating experience. In addition, trickling filter is easy to operate and maintain compared to other identified alternatives. Power requirements for trickling filters are similar or less than the identified alternatives.

Rotating biological contactors have received the second highest combined benefit score. Rotating biological contactors are covered to eliminate aerosol pollution. Although this increases capital cost of the facility which minimizes air quality concerns. This added feature contributed to the total benefit score of rotating biological contactor thereby making it second highest scoring technology.

Both trickling filters and rotating biological contactors are recommended as secondary treatment technology for further evaluation (cost will be incorporated in the final evaluation).

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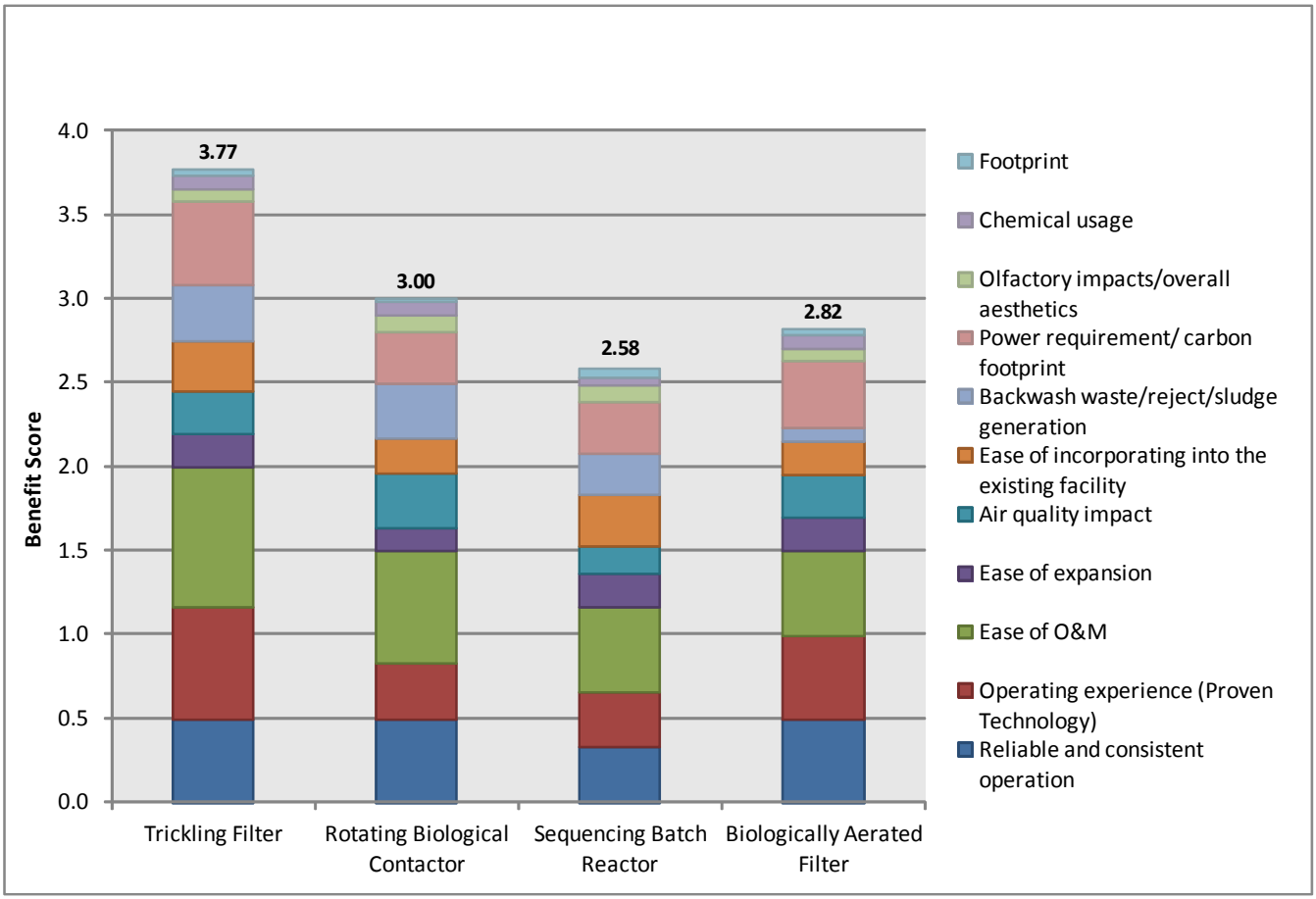
Exhibit 17 SMART Analysis Results for Secondary Treatment Alternatives



Filtration Options

The SMART analysis for filtration options showed that the conventional down flow sand filters have the highest combined benefit score (3.75) among 9 filtration alternatives identified. Proven track record, performance and ease of operation of trickling filters placed conventional down flow sand filters into the top spot. The combined benefit scores for the membrane filtration and cloth disc filtration (3.63 and 3.60, respectively) are also high and very close to the top score. Membrane filtration is a high cost option, but provides superior and reliable treatment without need for chemical addition. Cloth media disc filters are very energy efficient, cost effective, compact and easy to operate option. Therefore, in addition to conventional down flow sand filters, membrane and cloth disc media filtration are recommend for further evaluation.

Ability of cloth disc media filters to reliably meet effluent quality objectives relies on feed water quality and chemical pretreatment. Therefore, a detailed analysis (particle size distribution, filterability with and without chemicals) should be performed to determine feasibility of cloth media disc filters as a filtration option for the MLTP. As this project progresses, CH2MHILL has coordinated with Aqua Aerobic the collection of samples from the SWTP secondary effluent for particle size distribution and filterability analysis.

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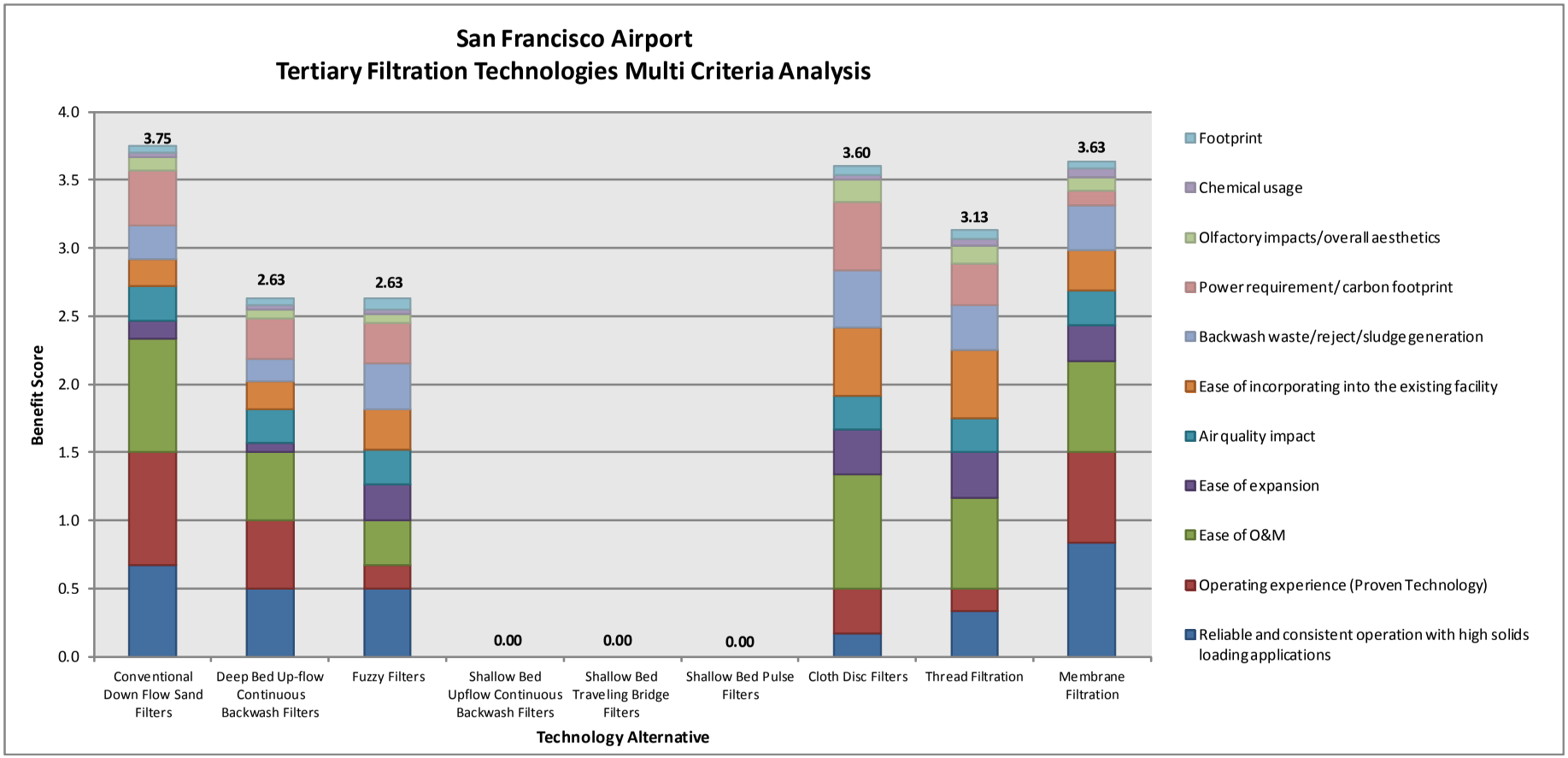
Exhibit 18 SMART Analysis Results for Filtration Alternatives



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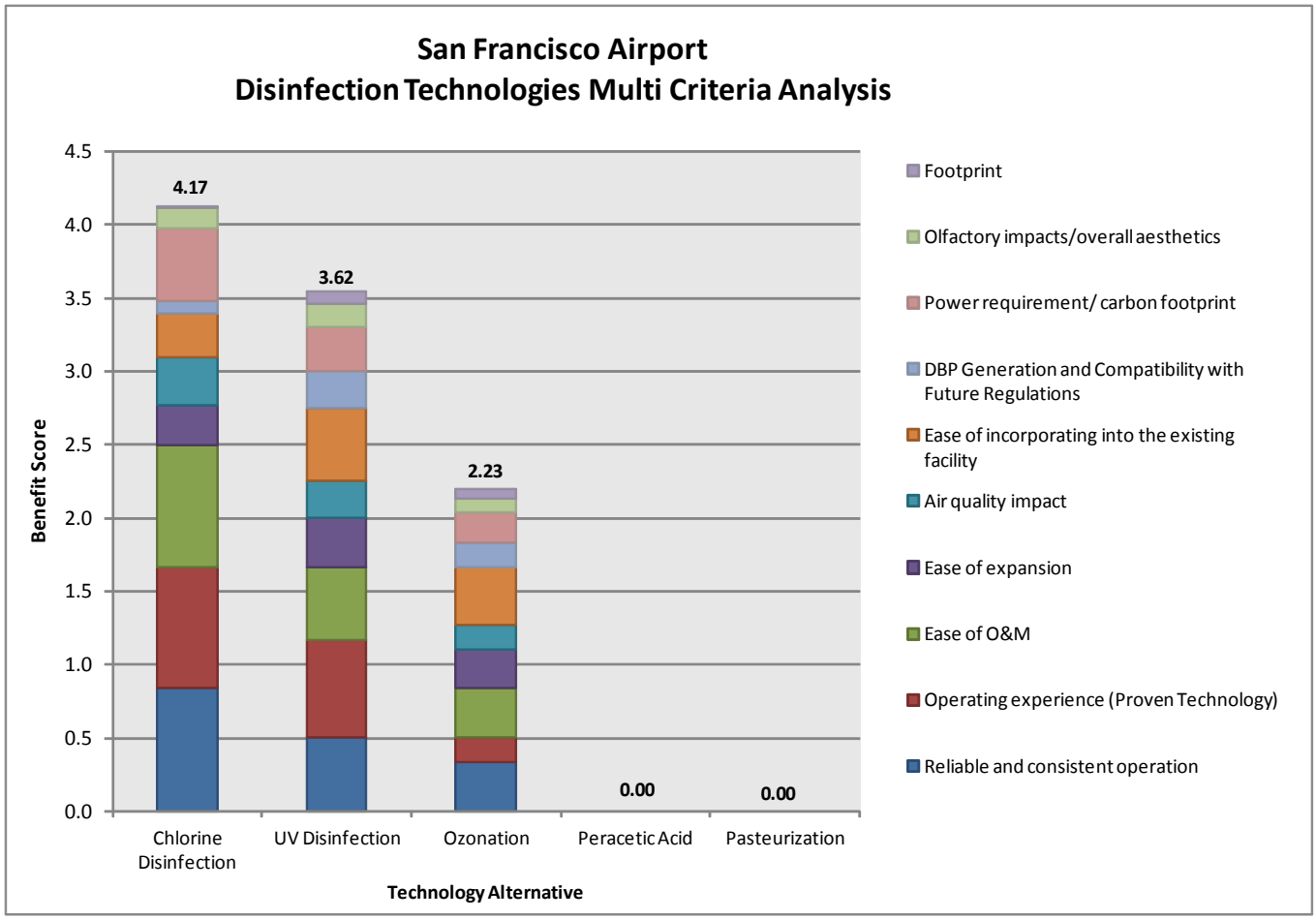
Disinfection Options

The SMART identified chlorine disinfection as the most suitable disinfection technology with a combined benefit score of 4.17 (Exhibit 19). Chlorine disinfection is proven, very effective and easy to operate and maintain. In addition, SFIA staff is currently operating chlorine disinfection system successfully. UV disinfection received the second highest score (3.62). One advantage of UV disinfection is that it does not generate DBPs. On the other hand, it has higher capital and O&M costs than chlorine disinfection. In addition, feed water quality has a strong impact on UV design and performance.

Ozonation is an approved disinfection process by the CDPH and provides effective disinfection with higher capital and O&M costs. Although ozonation has been used in the drinking water area for many years, there is no full-scale operating ozonation as a primary disinfection unit in Title 22 applications. Due to lack of operating experience, ozonation was eliminated for further consideration.

Pasteurization has recently been demonstrated in pilot scale. However, capital and O&M costs, long term performance, maintenance requirements yet to be determined. As a result, pasteurization was eliminated from further consideration.

Exhibit 19 SMART Analysis Results for Disinfection Alternatives



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3. Summary of Technology Evaluation

Based on the technology evaluation performed, the following technologies are recommended for further development as part of alternative overall treatment trains that will be evaluated in more detail and presented in forthcoming Technical Memorandum 4.

- Primary Treatment
  - Dissolved Air Flotation
- Secondary Treatment
  - Trickling Filter
  - Rotating Biological Contactor
- Tertiary Treatment
  - Conventional Down Flow Sand Filter
  - Membrane Filtration
  - Cloth Disc filtration
- Disinfection
  - Chlorination

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Appendix A

SMART Evaluation Results

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San Francisco Airport
Primary Treatment Technologies Multi Criteria Analysis

	Evaluation Criteria	Weighting Factor				Adjusted Weights			
			Oil Water Separator	DAF	Clarifier		Oil Water Separator	DAF	Clarifier
1	Reliable and consistent operation	10	2	5	2	0.17	0.33	0.83	0.33
2	Operating experience (Proven Technology)	10	4	5	5	0.17	0.67	0.83	0.83
3	Ease of O&M	10	2	4	4	0.17	0.33	0.67	0.67
4	Ease of expansion	4	4	4	1	0.07	0.27	0.27	0.07
5	Air quality impact	5	3	4	3	0.08	0.25	0.33	0.25
6	Ease of incorporating into the existing facility	6	2	3	1	0.10	0.20	0.30	0.10
7	Backwash waste/reject/sludge generation	5	1	4	4	0.08	0.08	0.33	0.33
8	Power requirement/ carbon footprint	6	4	3	4	0.10	0.40	0.30	0.40
9	Olfactory impacts/overall aesthetics	2	4	2	2	0.03	0.13	0.07	0.07
10	Chemical usage	1	5	2	2	0.02	0.08	0.03	0.03
11	Footprint	1	5	3	1	0.02	0.08	0.05	0.02
	Sum	60	**36**	**39**	**29**	1.00	**2.83**	**4.02**	**3.10**

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San Francisco Airport
Secondary Treatment Technologies Multi Criteria Analysis

	Evaluation Criteria	Weighting Factor	Score for Technology Alternatives					Adjusted Weights	Benefit Score				
			Trickling Filter	Rotating Biological Contactor	Sequencing Batch Reactor	Biologically Aerated Filter			Trickling Filter	Rotating Biological Contactor	Sequencing Batch Reactor	Biologically Aerated Filter	
1	Reliable and consistent operation	10	3	3	2	3		0.17	0.50	0.50	0.33	0.50	
2	Operating experience (Proven Technology)	10	4	2	2	3		0.17	0.67	0.33	0.33	0.50	
3	Ease of O&M	10	5	4	3	3		0.17	0.83	0.67	0.50	0.50	
4	Ease of expansion	4	3	2	3	3		0.07	0.20	0.13	0.20	0.20	
5	Air quality impact	5	3	4	2	3		0.08	0.25	0.33	0.17	0.25	
6	Ease of incorporating into the existing facility	6	3	2	3	2		0.10	0.30	0.20	0.30	0.20	
7	Backwash waste/reject/sludge	5	4	4	3	1		0.08	0.33	0.33	0.25	0.08	
8	Power requirement/ carbon footprint	6	5	3	3	4		0.10	0.50	0.30	0.30	0.40	
9	Olfactory impacts/overall aesthetics	2	2	3	3	2		0.03	0.07	0.10	0.10	0.07	
10	Chemical usage	1	5	5	3	5		0.02	0.08	0.08	0.05	0.08	
11	Footprint	1	2	1	3	2		0.02	0.03	0.02	0.05	0.03	
	Sum	60	**39**	**33**	**30**	**31**		1.00	**3.77**	**3.00**	**2.58**	**2.82**	

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San Francisco Airport
Tertiary Filtration Technologies Multi Criteria Analysis

	Evaluation Criteria	Weighting Factor	Score for Technology Alternatives									Adjusted Weights	Benefit Score								
			Conventional Down Flow Sand Filters	Deep Bed Up-flow Continuous	Fuzzy Filters	Shallow Bed Upflow Continuous	Shallow Bed Traveling Bridge Filters	Shallow Bed Pulse Filters	Cloth Disc Filters	Thread Filtration	Membrane Filtration		Conventional Down Flow Sand Filters	Deep Bed Up-flow Continuous	Fuzzy Filters	Shallow Bed Upflow Continuous	Shallow Bed Traveling Bridge Filters	Shallow Bed Pulse Filters	Cloth Disc Filters	Thread Filtration	Membrane Filtration
1	Reliable and consistent operation with high solids loading applications	10	4	3	3	0	0	0	1	2	5	0.17	0.67	0.50	0.50	0.00	0.00	0.00	0.17	0.33	0.83
2	Operating experience (Proven Technology)	10	5	3	1				2	1	4	0.17	0.83	0.50	0.17	0.00	0.00	0.00	0.33	0.17	0.67
3	Ease of O&M	10	5	3	2				5	4	4	0.17	0.83	0.50	0.33	0.00	0.00	0.00	0.83	0.67	0.67
4	Ease of expansion	4	2	1	4				5	5	4	0.07	0.13	0.07	0.27	0.00	0.00	0.00	0.33	0.33	0.27
5	Air quality impact	5	3	3	3				3	3	3	0.08	0.25	0.25	0.25	0.00	0.00	0.00	0.25	0.25	0.25
6	Ease of incorporating into the existing facility	6	2	2	3				5	5	3	0.10	0.20	0.20	0.30	0.00	0.00	0.00	0.50	0.50	0.30
7	Backwash waste/reject/sludge	5	3	2	4				5	4	4	0.08	0.25	0.17	0.33	0.00	0.00	0.00	0.42	0.33	0.33
8	Power requirement/ carbon footprint	6	4	3	3				5	3	1	0.10	0.40	0.30	0.30	0.00	0.00	0.00	0.50	0.30	0.10
9	Olfactory impacts/overall aesthetics	2	3	2	2				5	4	3	0.03	0.10	0.07	0.07	0.00	0.00	0.00	0.17	0.13	0.10
10	Chemical usage	1	2	2	2				2	3	4	0.02	0.03	0.03	0.03	0.00	0.00	0.00	0.03	0.05	0.07
11	Footprint	1	3	3	5				4	4	3	0.02	0.05	0.05	0.08	0.00	0.00	0.00	0.07	0.07	0.05
	Sum	60	**36**	**27**	**32**	**0**	**0**	**0**	**42**	**38**	**38**	1.00	**3.75**	**2.63**	**2.63**	**0.00**	**0.00**	**0.00**	**3.60**	**3.13**	**3.63**

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San Francisco Airport
Disinfection Technologies Multi Criteria Analysis

	Evaluation Criteria	Weighting Factor	Score for Technology Alternatives					Adjusted Weights	Benefit Score				
			Chlorine Disinfection	UV Disinfection	Ozonation	Pasteurization			Chlorine Disinfection	UV Disinfection	Ozonation	Pasteurization	
1	Reliable and consistent operation	10	5	3	2			0.17	0.83	0.50	0.33	0.00	
2	Operating experience (Proven Technology)	10	5	4	1	0		0.17	0.83	0.67	0.17	0.00	
3	Ease of O&M	10	5	3	2			0.17	0.83	0.50	0.33	0.00	
4	Ease of expansion	4	4	5	4			0.07	0.27	0.33	0.27	0.00	
5	Air quality impact	5	4	3	2			0.08	0.33	0.25	0.17	0.00	
6	Ease of incorporating into the existing facility	6	3	5	4			0.10	0.30	0.50	0.40	0.00	
7	DBP Generation and Compatibility with Future Regulations	5	1	3	2			0.08	0.08	0.25	0.17	0.00	
8	Power requirement/ carbon footprint	6	5	3	2			0.10	0.50	0.30	0.20	0.00	
9	Olfactory impacts/overall aesthetics	2	4	5	3			0.03	0.13	0.17	0.10	0.00	
10	Chemical usage	1	2	4	2			0.02	0.03	0.07	0.03	0.00	
11	Footprint	1	1	5	4			0.02	0.02	0.08	0.07	0.00	
	Sum	60	**39**	**43**	**28**	**0**		1.00	**4.17**	**3.62**	**2.23**	**0.00**	

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Appendix E.4
Technical Memorandum 4

DRAFT TECHNICAL MEMORANDUM 4 CH2MHILL®

San Francisco International Airport Treatment Technology Selection

PREPARED FOR: San Francisco International Airport

PREPARED BY: CH2M HILL

DATE: May 10, 2012

Introduction and Objectives

The City and County of San Francisco owns and operates the Mel Leong Treatment Plant (MLTP) through the San Francisco International Airport (SFIA). The MLTP includes an Industrial Wastewater Treatment Plant (IWTP) and a Sanitary Wastewater Treatment Plant (SWTP). The IWTP treats industrial wastewater as well as first flush, storm water runoff from airport facilities and the SWTP treats the sanitary sewer from the airport facilities. The effluent from both facilities (IWTP and SWTP) is routed to either the effluent pump station or the water reclamation pump station. The combined effluent is pumped to the North Bayside System Unit outfall, where the treated water is combined with effluent from South San Francisco, Millbrae, and Burlingame. Dechlorination takes place in the shared outfall before the effluent is discharged in to the Bay. The effluent from the water reclamation pump station is used within the Airport for irrigation purposes under a restricted use permit. The sludge from both facilities is conveyed to on-site sludge drying beds for dewatering and drying.

Technical Memorandum 3 (TM3) entitled "San Francisco International Airport Evaluation of Treatment Technology Alternatives" used multi criteria analysis to short list the following treatment technology alternatives based on the highest combined benefit scores received.

Treatment Technology Alternatives for IWTP Upgrade:

- Primary treatment: dissolved air flotation
- Secondary treatment: (1) trickling filters and (2) rotating biological contactors (RBCs)

Treatment Technology Alternatives for Tertiary Treatment for Reuse:

- Filtration: (1) conventional down flow sand filtration, (2) cloth disc filtration and (3) membrane filtration
- Disinfection: chlorine disinfection (initial evaluation also identified UV disinfection as an alternative. However, this option was eliminated from further consideration due to unfamiliarity of plant operators with UV disinfection and operation and maintenance concerns as a result of chemicals used in the upstream treatment process (i.e., pretreatment of filtration).

In this TM (TM 4), the short listed technologies presented above (except for DAF which was pre-selected in TM3) are further evaluated using a 25 year life cycle cost (LCC) analysis. The objective of TM 4 is to develop benefit-to-cost ratios for the treatment alternatives using SMART (CH2MHILL's proprietary decision science model) to select a technology in each category based on the highest calculated benefit-to-cost ratios.

TM 4 contains the following sections:

1. Basis of Cost Estimates for LCC Analysis
2. Evaluation of Primary and Secondary Treatment Technology Alternatives for IWTP Upgrade
    - Background
    - Design Criteria, Sizing and Cost Estimate Assumptions
    - Planning Level Cost Estimate for the Treatment Alternatives
    - Benefit to Cost Ratios for the Treatment Alternatives

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3. Evaluation of Tertiary Treatment Alternatives
    - Background
    - Design Criteria, Sizing and Cost Estimate Assumptions
    - Planning Level Cost Estimate for the Treatment Alternatives
    - Benefit to Cost Ratios for the Treatment Alternatives

4. Summary of the Treatment Technology Selection

1. Basis of Cost Estimates for Life Cycle Cost Analysis

Cost estimates for the technology alternatives were developed by obtaining budgetary-level equipment costs from equipment suppliers and calculating facility costs using CH2M HILL's cost-estimating methodology (CPES) for projects of similar type and size.

The cost estimates developed for this analysis provide a relative comparison of the treatment alternatives and are considered order-of-magnitude estimates. An order-of-magnitude cost estimate is defined as "an approximate estimate made without detailed engineering data." The Association for the Advancement of Cost Engineering (AACE) International defines order-of-magnitude costs as *Class 5* cost estimates without detailed engineering data. This estimate is prepared based on limited information, where little more than proposed plant type, its location, and the capacity are known, where preliminary engineering is from 0% to 2% complete. Strategic planning purposes include, but are not limited to, market studies, assessment of viability, evaluation of alternate schemes, project screening, location and evaluation of resource needs and budgeting, and long-range capital planning. Examples of estimating methods used would include cost/capacity curves and factors, scale-up factors, and parametric and modeling techniques. Typically, little time is expended in the development of this estimate. The expected accuracy ranges for this class of estimate are –20% to –50% on the low range side and +30% to +100% on the high range side.

The cost estimates prepared, which include any resulting conclusions on project financial or economic feasibility or funding requirements, have been prepared for guidance in project evaluation and implementation from the information available at the time of the estimate. The final costs of the project and resulting feasibility will depend on actual labor and material costs, competitive market conditions, actual site conditions, final project scope, implementation schedule, continuity of personnel and engineering, and other variable factors. Therefore, the final project costs will vary from the estimate presented here. Because of these factors, project feasibility, benefit/cost ratios, risks, and funding needs must be carefully reviewed prior to making specific financial decisions or establishing project budgets to help ensure proper project evaluation and adequate funding.

The subtotal of capital cost includes the following contractor markups:

- Bond and insurance: 5 percent
- Contractors overheads: 10 percent
- Contractor profit: 6 percent

The following contingencies (as an additional percentage of the sub total) were included in the facility costs:

- Project contingency: 25 percent
- Engineering and administrative fees: 20 percent

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The chemical and power unit costs were obtained from similar projects in San Francisco area and presented in Table 1-1. These unit costs were used for comparing treatment alternatives on a 25-year LCC basis at 5.0 percent discount rate. LCC is the total cost of ownership of equipment and technology, including its cost of acquisition and operations and maintenance (O&M). It takes into account the costs associated with consumables (chemicals, energy) and parts replacement (i.e., membrane replacement, cloth media replacement), and is an effective, unbiased method to choose the most cost-effective alternative from a series of alternatives.

TABLE 1-1
Electricity Unit Cost and Chemical Unit Costs

Item	Unit Cost
Electricity, $/kWh	0.12
Alum (Dry), $/lb	0.35
Citric Acid (50%), $/gal	5.99
Sodium hypochlorite (12.5%), $/gal	1.20
Sodium bisulfite (25%), $/gal	2.25
Sodium hydroxide (25%), $/gal	2.38

2. Evaluation of Primary and Secondary Treatment Technology Alternatives for IWTP Upgrade

Background

The wastewater flows vary and show significant daily and seasonal fluctuations. While the summer flows are projected to be approximately less than 0.1 million gallons per day (mgd), flows as high as 1.7 mgd have been recorded during wet periods. Most units of the IWTP need to be replaced. The proposed process schematic showing existing and new units can be found in Appendix A. For primary treatment, TM3 identified dissolved air flotation (DAF) as the most suitable primary treatment option compared to oil/water separators or primary clarifiers; primarily because it is a technology that can easily handle emulsions and flow variations. In this TM a cost comparison is unnecessary for primary treatment; therefore, no design criteria and no cost estimates for primary treatment are provided. Furthermore, the common chlorination step is not included in this secondary treatment process comparison. Primary treatment and chlorination facilities will be developed in the next process development stage of the project.

The secondary treatment will be designed to treat peak flows and variable loads. In addition, the two short listed treatment alternatives should meet project objectives in a cost effective manner. The project water quality objectives are two-fold:

1. The treated wastewater should fully satisfy National Pollutant Discharge Elimination System (NPDES) permit limit for BOD, TSS and other specified constituents
2. The treated wastewater should satisfy Title 22 tertiary recycled water criteria when coupled with filtration and disinfection. This is required based on SFIA staff's interest to minimize NPDES discharges while maximizing water reuse

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Design Criteria, Sizing Unit Treatment Processes and Cost Estimate Assumptions

To satisfy redundancy requirements by SFIA operations, two parallel trains of equipment will be provided for the IWTP. Table 2-1 summarizes key design criteria used for sizing the new secondary treatment unit processes.

TABLE 2-1
Design Criteria for Secondary treatment Unit Processes

Criteria	Description
Design flow for the IWTP, mgd	1.7
SECONDARY TREATMENT	
Trickling Filters	
Number of filters	2
Design feed flow per train, mgd	0.85
Filter diameter, ft	66
Media filter height, ft	10 (based on existing facility design)
Design Recirculation rate per unit, mgd	1.23 (based on 0.25 gpm/sf)
Rotating Biological Contactors	
Number of contactors per train	3 (6 total for 2 trains)
Design feed flow per train, mgd	0.85
Total surface area per unit, sf	114,750 (688,500 sq.ft. total)
Soluble BOD loading on stage 1, lbs BOD/1000sf/day	2.07
Secondary Clarifiers	
Number of clarifiers	2
Design feed flow per train, mgd	0.85
Maximum surface overflow rate, gpd/sf	1,060
Tank diameter, ft	32
Sidewater depth, ft	15

Cost Estimate Summary for the Secondary Treatment Alternatives

The design criteria presented in Table 2-1 were used to develop capital, O&M and LCC for the two short listed secondary treatment alternatives. The capital cost estimate was prepared as described under Section 1.

Table 2-2 summarizes capital, O&M and LCCs for the two secondary treatment alternatives evaluated. Cost estimate details for the secondary treatment alternatives are presented in Appendix B. As indicated in the process schematic in Appendix A, both the trickling filters and the RBC units are followed by a secondary clarifier. Consequently, the secondary clarifier costs are included in each secondary treatment alternative. The costs displayed in Table 2-2 include the associated costs with the secondary clarifier and sludge recirculation pump station. Since the chlorination basin and tertiary treatment pump station and associated costs will be identical for the two alternatives evaluated, these costs are not included in Table 2-2.

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Key O&M assumptions common to each alternative include the following:

- Annual O&M cost is based on annual average flow of approximately 0.58 mgd.
- No additional personnel are needed to operate and maintain the facilities in each treatment alternative.
- Facility runs 24 hours per day, 365 days per year.
- One percent of the direct cost is assigned for maintenance.

TABLE 2-2
Capital, O&M and LCCs for the Two Secondary Treatment Alternatives

Item	Trickling Filters + Secondary Clarifier	RBCs + Secondary Clarifier
Total Capital Cost, $	**7,111,000**	**7,667,000**
Total O&M Cost, $/year	**49,000**	**52,000**
Life Cycle Cost (LCC), $	**7,802,000**	**8,400,000**

Alternative costs are relatively close. However, both the capital and O&M costs associated with the trickling filter are lower, resulting in a lower LCC of about $600,000, compared to the RBC's LCC. It should be realized that the cost comparison assumed concrete tanks for both trickling filters and RBCs. The use of steel tank may reduce the capital cost for the trickling filters. The use of steel tanks in lieu of concrete tanks in trickling filters will be further evaluated in the fort coming task of the project.

Benefit to Cost (B/C) Ratios

The TM 3 short listed the top two secondary treatment alternatives based on the total benefit scores. The total benefit scores were 3.77 and 3.00 for trickling filters and RBCs, respectively. In this section, the total benefit scores are coupled with the LCCs to estimate benefit to cost ratios for the two alternatives. Figure 2-1 shows the estimated benefit to cost (B/C) ratios and LCC for the two alternatives. The cost values on B/C in Figure 2-1 refer to LCCs and are as $1 million.

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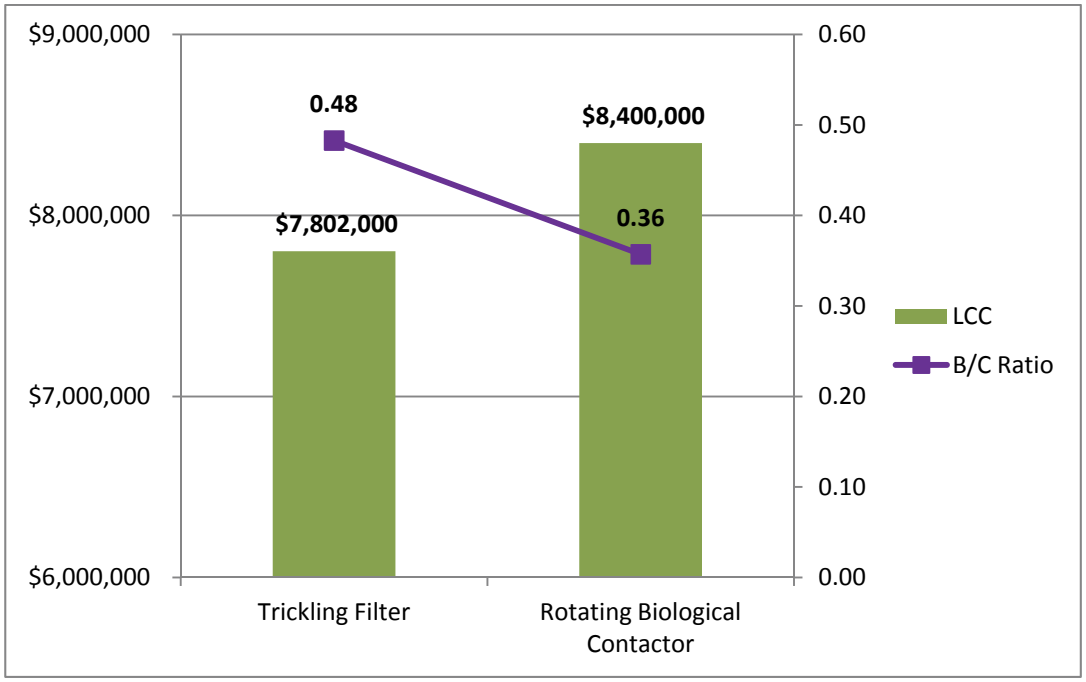
FIGURE 2-1
B/C Ratios and LCCs for the Two Secondary Treatment Alternatives



Based on the results shown in Figure 2-1, trickling filters exhibited the highest cost to benefit ratio (0.48 compared to 0.36) of the two alternatives evaluated and identified as the most favorable secondary treatment alternative. In addition, trickling filters are the recommended secondary treatment alternative because:

- They are effective for treating wastewater flows and loads exhibiting significant fluctuations as in the IWTP.
- They are easy to operate and maintain
- SFIA operators have good operating experience with trickling filter

3. Evaluation of Tertiary Treatment Technology Alternatives for Reuse

Background

The proposed tertiary treatment plant will produce up to 1.0-mgd disinfected tertiary recycled water for unrestricted non-potable uses including landscape irrigation and urban reuse. California Department of Public Health (CDPH) defines disinfected tertiary recycled water as “oxidized, filtered and subsequently disinfected wastewater” that meets the criteria presented in Table 3-1.

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TABLE 3-1
Water Quality Requirement for Disinfected Tertiary Recycled Water

Parameter	Requirement
Organic material	Wastewater has to be properly oxidized under all conditions. " 'Oxidized wastewater' means wastewater in which the organic matter has been stabilized, is non-putrescible, and contains dissolved oxygen." All biological treatment unit processes shall be provided with reliability features such as "alarms and multiple biological treatment units capable of producing oxidized wastewater with one unit not in operation."
Turbidity (if treated through natural undisturbed soils or a bed of filter; i.e., dual or mixed media, upflow or pressure filtration systems, traveling bridge automatic filters)	Average of 2 NTU within 24-hr period 5 NTU not more than 5% of the time during 24-hr period Less than 10 NTU at all times
Turbidity (if treated through microfiltration, ultrafiltration, nanofiltration, or reverse osmosis membranes)	Average of 0.1 NTU within 24-hr period 0.2 NTU not more than 5% of the time during 24-hr period Less than 0.5 NTU at all times
Disinfection	A chlorine disinfection process following filtration that provides a CT (the product of total chlorine residual and modal CT measured at the same point) value of not less than 450 milligram-minutes per liter at all times with a modal CT of at least 90 minutes, based on peak dry weather design flow; or A disinfection process that, when combined with the filtration process, has been demonstrated to inactivate and/or remove 99.999 percent of the plaque forming units of F-specific bacteriophage MS2, or polio virus in the wastewater.
Total coliform bacteria	2.2 MPN per 100 mL per sample, median reading not to exceed over any 7-day continuous period 23 MPN per 100 mL per sample, not to occur more than once within 30 days 240 MPN per 100 mL in any sample

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Design Criteria, Sizing Unit Treatment Processes and Cost Estimate Assumptions

As discussed in TM 3, the tertiary treatment plant will be designed to handle both ITWP and SWTP secondary effluent. However, as noted in TM3, the SWTP secondary effluent will be used as the primary source for the tertiary treatment. Furthermore, the quality of the SWTP secondary effluent is such that it controls the design of the tertiary treatment plant design. The SWTP secondary effluent has an average turbidity of 11 NTU with periodic excursion exceeding 20 NTU, which requires full pretreatment (coagulation, flocculation and solids removal) to meet Title 22 recycled water turbidity requirements summarized in Table 3-1. Because membrane filtration is an absolute barrier to solids, chemical pretreatment for solids removal is not required to meet turbidity requirements for membrane filtration. To satisfy redundancy criteria of Title 22 recycled water regulations, two treatment trains will be provided where the TTP will satisfy average recycled water demand as the largest train out of service. Table 3-2 summarizes key design criteria used for sizing the tertiary treatment unit processes.

TABLE 3-2
Design Criteria for Treatment Unit Processes

Criteria	Description
Annual average flow, mgd	0.64
Maximum design flow, mgd	1.00
FILTRATION TECHNOLOGIES	
Conventional Deep Bed Sand Filters	
Design feed flow, mgd	1.10 (assumes 10% backwash waste)
Design filtrate flow, mgd	1.00
Pretreatment	
Coagulation	
Number of filters	2
Rapid mix velocity gradient, sec^{-1}	2,000
Minimum water temperature, ^{o}C	15
Projected alum dose, mg/L	10 (for comparison purpose only, other chemicals including iron salts and other form of aluminum salts can be used for the same purpose. The dosed to be verified during full design)
Flocculation	
Number of units	2
Number of flocculation stages, each unit	3
Detention time, min, each unit	15
Velocity gradient in the first stage, sec^{-1}	60
Velocity gradient in the second stage, sec^{-1}	40
Velocity gradient in the third stage, sec^{-1}	20
Inclined Plate Settling	
Number of settling basins	2
Maximum hydraulic loading rate, each settler, gpm/sq-ft	0.4
Plate angles, degrees	55
Projected effluent turbidity, NTU	<5
Sand Filters	
Type of media	Mono media sand
Minimum media depth, inches	60
Number of Filters	2

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Maximum Design Surface Overflow Rate at 1.0 mgd, gpm/ft^2	6 (with 1 train out of service)
Projected Backwash Waste (% of the feed)	5-10
Cloth/Disc Media Filters	
Design Feed Flow, mgd	1.03 (assumes 13% backwash waste)
Design Filtrate Flow, mgd	1.00
Pretreatment	
Coagulation	
Number of units	2
Rapid mix velocity gradient, sec-^{1}	2,000
Minimum water temperature, ^{o}C	15
Projected alum dose, mg/L	10 (for comparison purpose only, other chemicals including iron salts and other form of aluminum salts can be used for the same purpose. The dosed to be verified during full design)
Flocculation	
Number of untis	2
Number of flocculation stages, each unit	3
Detention time, min, each unit	15
Velocity gradient in the first stage, sec-^{1}	60
Velocity gradient in the second stage, sec-^{1}	40
Velocity gradient in the third stage, sec-^{1}	20
Inclined Plate Settling	
Number of settling basins	2
Maximum hydraulic loading rate, each settler, gpm/sq-ft	0.4
Plate angles, degrees	55
Projected effluent turbidity, NTU	<5
Cloth Disc Filters	
Type of media	Cloth/fabric
Nominal pore size of the media, inches	10-11
Number of filter tanks	2
Maximum filtration rate at 1.0 mgd, gpm/ft^2	6 (with 1 train out of service)
Projected backwash waste (% of the feed)	1-3
Membrane Filtration (Based on Pressurized Systems)	
Membrane Pretreatment-Self Cleaning Strainer	
Design filtrate flow, mgd	1.05
Clean strainer pressure drop at the design flow, psig	1
Projected backwash waste (% of the feed)	0.5
Membrane System	
Design feed flow, mgd	1.05
Design filtrate flow, mgd	1.0
No. of membrane trains	2
Design flux with one train out of service, gal per day per square foot (gpd/sq-ft)	25
Recovery, %	95
Maximum TMP, psi	40
Minimum spare module space, %	10
DISINFECTION TECHNOLOGIES	
Chlorine Disinfection	

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Design feed flow, mgd	1.0
Minimum hydraulic detention time at 1.0 mgd design flow, min	120
Minimum CT requirement, mg*min/L	450
Typical chlorine dose, mg/L	10 with membrane filtrations 12 with other filtration technologies

Cost Estimate Summary for the Tertiary Treatment Alternatives

The design criteria presented in Table 3-2 and unit costs presented in Table 1-1 were used to develop capital, O&M and LCC for three short listed tertiary filtration alternatives. Cost estimate details for the tertiary filtration alternatives are presented in Appendix B. As previously discussed, with the exception of membrane filtration, the conventional down flow sand filters and cloth media filters require coagulation, flocculation, and settling prior to filtration to reduce solids to a manageable levels (<5-7 NTU) in filtration influent to reliably meet Title 22 recycled water quality requirements for turbidity. Table 3-3 summarizes capital, O&M and LCCs for the three filtration alternatives evaluated. Since the chlorine disinfection and recycled water storage and associated costs will be identical for the three filtration alternatives evaluated, these costs are not included in Table 3-3.

Key O&M assumptions common to each alternative include the following:

- Annual O&M cost is based on annual average flow of approximately 0.64 mgd.
- No additional personnel are needed to operate and maintain the facilities in each treatment alternative.
- Facility runs 24 hours per day, 365 days per year.
- One percent of the direct cost is assigned for maintenance.
- Backwash waste from the filtration facilities will be returned to head of the STP. Costs associated with backwash flow conveyance were not included for alternative comparison.
- The major equipment including membrane filtration system and associated chemical clean in place system, cloth disc filter system and chemical feed facilities are housed under a canopy. The installed canopy unit cost is $40 per square foot.
- MF membrane will be replaced every 10 years and MF module replacement costs $2,900 per module based on Pall Aria membranes provided by Pall Inc. It should be noted that Pall Aria Membranes are used for the evaluation purpose only. Other CDPH approved membrane products supplied by GE Water Technologies, Siemens, Toray, Hydranautics, etc. are also qualified and can be considered in this project.
- Cloth/disc media capital and O&M costs projections are based on Aqua Aerobic's Aquadisk system and obtained from Aqua Aerobic. Chlorine tolerant polyester (PET) cloth media was assumed in this evaluation. It should be noted that Aqua Aerobic disc system is used for the evaluation purpose only. Other CDPH approved cloth/disc products supplied by Siemens, Kruger, etc. are also qualified and can be considered in this project.

TABLE 3-3
Capital, O&M and LCCs for the Three Filtration Alternatives

Item	Membrane Filtration	Cloth Disc Media Filtration	Conventional Sand Media Filtration
Total Capital Cost, $	**3,785,000**	**4,556,000**	**5,237,000**
Total O&M Cost, $/year	**85,000**	**46,000**	**49,000**
Life Cycle Cost (LCC), $	**4,983,000**	**5,204,000**	**5,928,000**

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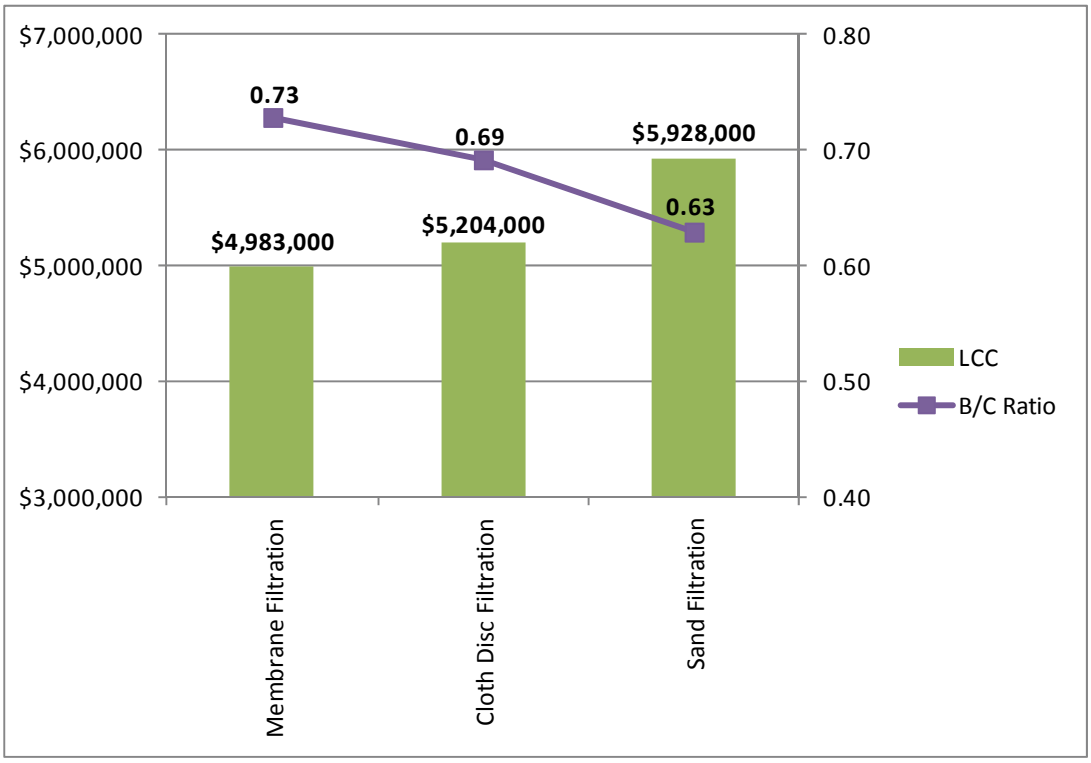
Even though membrane filtration usually costs higher than other filtration alternatives, the need for pretreatment (i.e., coagulation, flocculation and settling) for the cloth disc media and conventional sand media filtration alternatives has increased the capital costs for both cloth disc and conventional sand media filtration. Despite the highest O&M cost (high cost is primarily associated with membrane replacement cost), membrane filtration has the lowest LCCs among the three filtration alternatives evaluated.

Benefit to Cost (B/C) Ratios

The TM 3 short listed the top three filtration alternatives based on the total benefit scores. The total benefit scores were 3.75, 3.63 and 3.60 for conventional sand filtration, membrane filtration and cloth disc media filtration, respectively. In this section, the total benefit scores are coupled with the LCCs to estimate benefit to cost ratios for the filtration alternatives (Per SFIA staff' input, the chlorine disinfection was chosen as the disinfection process in the tertiary treatment facility. Therefore, the benefit to cost ratio only covered filtration alternatives). Figure 3-1 shows the estimated benefit to cost (B/C) ratios and LCC for the three filtration alternatives. The cost values on B/C in Figure 3-1 refer to LCCs and are as $1 million.

FIGURE 3-1
B/C Ratios and LCCs for the Three Filtration Alternatives



Based on results shown in Figure 3-1, membrane filtration exhibited the highest cost to benefit ratio (0.73) of the alternatives evaluated and identified as the most favorable filtration alternative. Membrane filtration has the lowest capital costs among the evaluated technologies, it offers very reliable and robust performance, fully satisfies treated water quality objectives without chemical addition and regardless of the feed water quality.

Cloth disc filtration and sand media filtration cannot reliably meet the water quality requirements without full pretreatment which substantially increase capital cost thereby yielding relatively low B/C ratio.

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4. Summary of the Technology Selection

A three-step decision process has been utilized to select secondary treatment and tertiary filtration alternatives.

The secondary treatment technology selection and comparison are based on 1.7 mgd treatment capacity. The SMART (CH2MHILL's proprietary decision science model) identified and short listed the top two alternatives (trickling filters and rotating biological contactors) based on the combined benefit scores in TM 3. Capital, O&M and LCCs were then estimated for each technology. The benefit scores were then divided by the net present value cost to develop a benefit-to-cost ratio that was then used as a quantitative means of ranking the technologies and selecting the most favorable filtration alternative.

Trickling filters exhibited the highest cost to benefit ratio (0.48) and was identified as the most favorable secondary treatment alternative. Trickling filters have been a choice for treating wastewater flows and loads exhibiting significant fluctuations as in IWTP. SFIA is currently operating trickling filter at the IWTP which all together maximized the score for operating experience.

The tertiary treatment filtration technology selection and comparison are based on 1.0 mgd treatment capacity to produce an average recycled water of 0.64 mgd. The SMART identified and short listed the top three filtration alternatives (membrane filtration, cloth disc filtration and conventional sand media filtration) based on the combined benefit scores in TM 3. Capital, O&M and LCCs were then estimated for each technology. The benefit scores were then divided by the net present value cost to develop a benefit-to-cost ratio that was then used as a quantitative means of ranking the technologies and selecting the most favorable filtration alternative.

Membrane filtration exhibited the highest cost to benefit ratio (0.73) and identified as the most favorable filtration alternative. Even though membrane filtration has the highest annual O&M costs ($85,000/year) among the three evaluated technologies, it offers very reliable and robust performance, fully satisfies treated water quality objectives without need for chemical pretreatment and independent of feed water quality. Membrane filtration is recommended for the implementation at the SFIA.

Membrane filtration will be coupled with chlorine disinfection to produce disinfected tertiary treated recycled water at the SFIA. One option is to use a separate chlorine contact basin and recycled water storage tank. The other option is to build a recycled water storage tank with an integrated chlorine contact basin. The later option is expected to lower the capital cost and offers some flexibility as chlorine contact basin is expended in future. These options will be further evaluated during design phase of the project.

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Appendix A

Process Flow Diagram for IWTP

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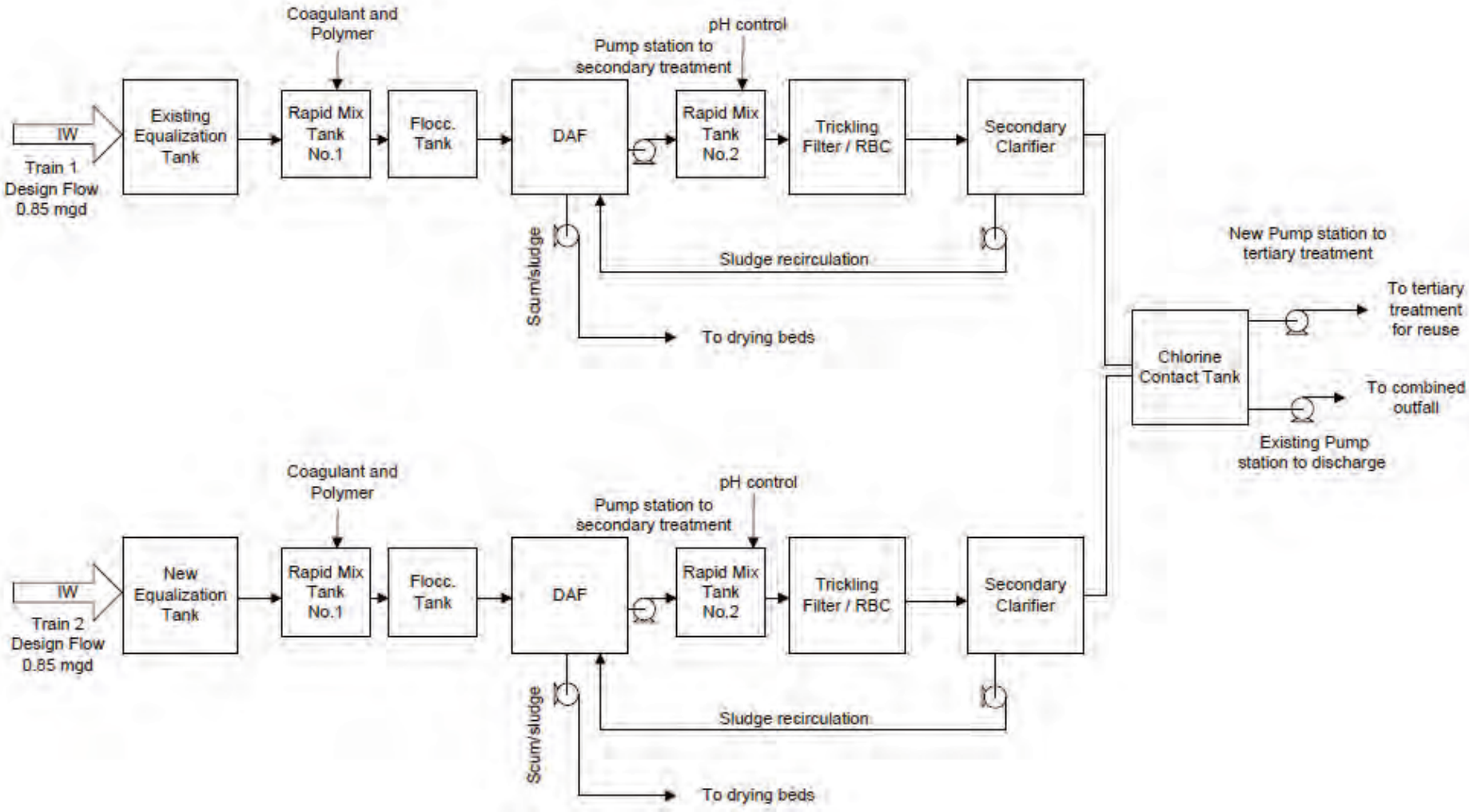

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Appendix B

Cost Estimate Details

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TABLE B-1
Capital, O&M and LCCs for the Two Secondary Treatment Alternatives

Item	Trickling Filters + Secondary Clarifier	RBCs + Secondary Clarifier
Capital Cost		
Siteworks	162,000	96,000
Concrete	1,170,000	315,000
Canopy/Building	318,000	121,000
Metals	75,000	75,000
Equipment	1,191,000	2,485,000
I&C	195,000	235,000
Mechanical	315,000	345,000
Electrical	157,000	192,000
Subtotal, $	**3,583,000**	**3,864,000**
Siteworks Allowance	108,000	116,000
Yard Piping Allowance	180,000	194,000
Electrical Allowance	180,000	194,000
Direct Cost with Allowances, $	**4,051,000**	**4,368,000**
Contractor's Markups		
Bonds and Insurance (5%)	203,000	219,000
Contractor's Overhead (10%)	406,000	437,000
Contractor's Profit (6%)	244,000	263,000
Total Markups	853,000	919,000
Sub Total with Markups, $	**4,904,000**	**5,287,000**
Project Contingency, 25% of the Subtotal with Markups, $	1,226,000	1,322,000
Engineering, Legal and Administrative Fees, 20% of the Sub Total with Markups, $	981,000	1,058,000
Total Capital Cost, $	**7,111,000**	**7,667,000**
O&M Cost		
Power, $/year	12,000	12,000
Maintenance, $/year	37,000	40,000
Total O&M Cost, $/year	**49,000**	**52,000**
Life Cycle Cost (LCC), $	**7,802,000**	**8,400,000**

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TABLE B-2
Capital, O&M and LCCs for the Three Filtration Alternatives

Item	Membrane Filtration	Cloth Disc Media Filtration	Conventional Sand Media Filtration
Capital Cost			
Siteworks	20,000	73,000	75,000
Concrete	63,000	158,000	522,000
Canopy/Building	160,000	206,000	344,000
Metals	10,000	47,000	51,000
Equipment	1,433,000	1,394,000	915,000
I&C	60,000	105,000	276,000
Mechanical	80,000	140,000	322,000
Electrical	80,000	173,000	134,000
Subtotal, $	**1,906,000**	**2,296,000**	**2,639,000**
Siteworks Allowance	58,000	69,000	80,000
Yard Piping Allowance	96,000	115,000	132,000
Electrical Allowance	96,000	115,000	132,000
Direct Cost with Allowances, $	**2,156,000**	**2,595,000**	**2,983,000**
Contractor's Markups			
Bonds and Insurance (5%)	108,000	130,000	150,000
Contractor's Overhead (10%)	216,000	260,000	299,000
Contractor's Profit (6%)	130,000	156,000	179,000
Total Markups	454,000	546,000	628,000
Sub Total with Markups, $	**2,610,000**	**3,141,000**	**3,611,000**
Project Contingency, 25% of the Subtotal with Markups, $	653,000	786,000	903,000
Engineering, Legal and Administrative Fees, 20% of the Sub Total with Markups, $	522,000	629,000	723,000
Total Capital Cost, $	**3,785,000**	**4,556,000**	**5,237,000**
O&M Cost			
Power, $/year	19,000	12,000	14,000
Consumables, $/year	32,000	2,000	0
Chemicals, $/year	14,000	7,000	7,000
Maintenance, $/year	20,000	25,000	28,000
Total O&M Cost, $/year	**85,000**	**46,000**	**49,000**
Life Cycle Cost (LCC), $	**4,983,000**	**5,204,000**	**5,928,000**

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San Francisco International Airport Recycled Water Project

Alternatives Analysis Report

June 15, 2009

Prepared for:
San Francisco Airport Commission

Prepared by:
Department of Public Works
Bureau of Engineering

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Executive Summary

ES.1 Project Background

San Francisco International Airport ("SFIA") has prepared a preliminary planning report to provide a strategy for implementing a recycled water project. Recycled water has been identified as a reliable and sustainable alternative water source that can reduce fresh water demand for irrigation and other non-potable purposes, and will reduce the level of rationing during drought periods.

ES.2 Alternative Analysis and Evaluation Approach

The alternative analysis and evaluation ("AAE") is a procedure used by the San Francisco Department of Public Works ("SFDPW") to select a preferred alternative for the implementation of capital improvement projects prior to preparation of a conceptual engineering report ("CER"). This alternative analysis report ("AAR") is the documentation of the AAE process.

The first step of the AAE was to review, update, and define recycled water demands developed in the 2008 recycled water planning phase (generated by SFIA engineering) to provide a basis for developing project design criteria for the treatment, storage, and distribution facilities. Table ES-1 presents a summary of all refined demand estimates for recycled water for the project. Figure ES.1 shows an overview of the potential recycled water users.

Table ES-1
Summary of Refined Demand Estimates

	Annual Avg (MGD)	Summer (May-Oct) Avg (MGD)	Winter (Nov-Apr) Avg (MGD)
Landscape Irrigation	0.10	0.16	0.04
Dual Plumbing	0.17	0.17	0.17
Cooling Towers - Airport	0.13	0.15	0.11
Cooling Towers - Cogen	0.15	0.15	0.15
MLTP	0.10	0.10	0.10
Total	**0.65**	**0.73**	**0.57**

Using the secondary effluent from the Mel Leong Treatment Plant ("MLTP"), two qualities of recycled water will be produced to meet the needs of the identified end users. Disinfected tertiary recycled water, meeting the requirements of Title 22 in the California Code of Regulations, would be used for landscaping irrigation. This water may also be used in industrial and municipal applications as well as dual plumbing systems. Water used for cooling towers at the United Airlines Cogeneration Facility and SFIA Central

Plant requires an advanced level of treatment because of the potential damage to cooling tower heat exchangers.

Currently, there are many treatment technologies available that will produce Title 22 quality water. In contrast, advanced treatment technologies for producing a higher quality of water are limited to reverse osmosis ("RO") or microfiltration and ultra-filtration membrane filtration ("MF/UF").

In order to identify the best combination of treatment technologies, site locations, and distribution system routing, a systematic and objective approach was used to analyze and rank the alternatives. This AAE process included:

- Development of statement of objectives.
- Initial screening of alternatives using boundary conditions.
- Establishment of evaluation criteria to measure relative benefits of alternatives.
- Assessment of net benefits of alternatives.
- Assessment of costs of alternatives.
- Assessment of benefit-to-cost ratios of alternatives.
- Identification of the programmatic alternative that offers the best value.

ES.3 Conclusions and Recommendations

Presented below are the preferred recommendations for each of the following objectives: tertiary and advanced treatment of recycled water, viable treatment locations, and distribution alternatives.

- Tertiary Treatment (Title 22 water)
    - Rapid-mix coagulation
    - Continuous backwash sand filtration
    - Ultraviolet irradiation disinfection
    - Sodium hypochlorite residual disinfection
- Advanced Treatment
    - Reverse Osmosis
    - Microfiltration/Ultra-filtration membrane filtration
- Treatment Locations
    - Mel Leong Treatment Plant
    - Parking Lot C near the SFIA/US-101 interchange
- Distribution Alternatives
    - Centralized/Decentralized treatment with centralized/decentralized supply

For this analysis, "centralized" is defined as tertiary/advanced treatment processes (and irrigation supply) being located at the same location, the MLTP. Conversely, "decentralized" is defined as the tertiary/advanced treatment (and irrigation supply) being in multiple locations. Since the tertiary treatment requires being near the MLTP secondary effluent, "decentralized" refers to the placement of advanced treatment facilities.

Thus, four scenarios were considered using the distribution alternatives for cost comparison. Table ES-2 summarizes the cost versus each scenario described below:

Scenario 1: Centralized treatment with centralized irrigation supply.
Scenario 2: Centralized treatment with decentralized irrigation supply.
Scenario 3: Decentralized treatment with centralized irrigation supply.
Scenario 4: Decentralized treatment with decentralized irrigation supply.

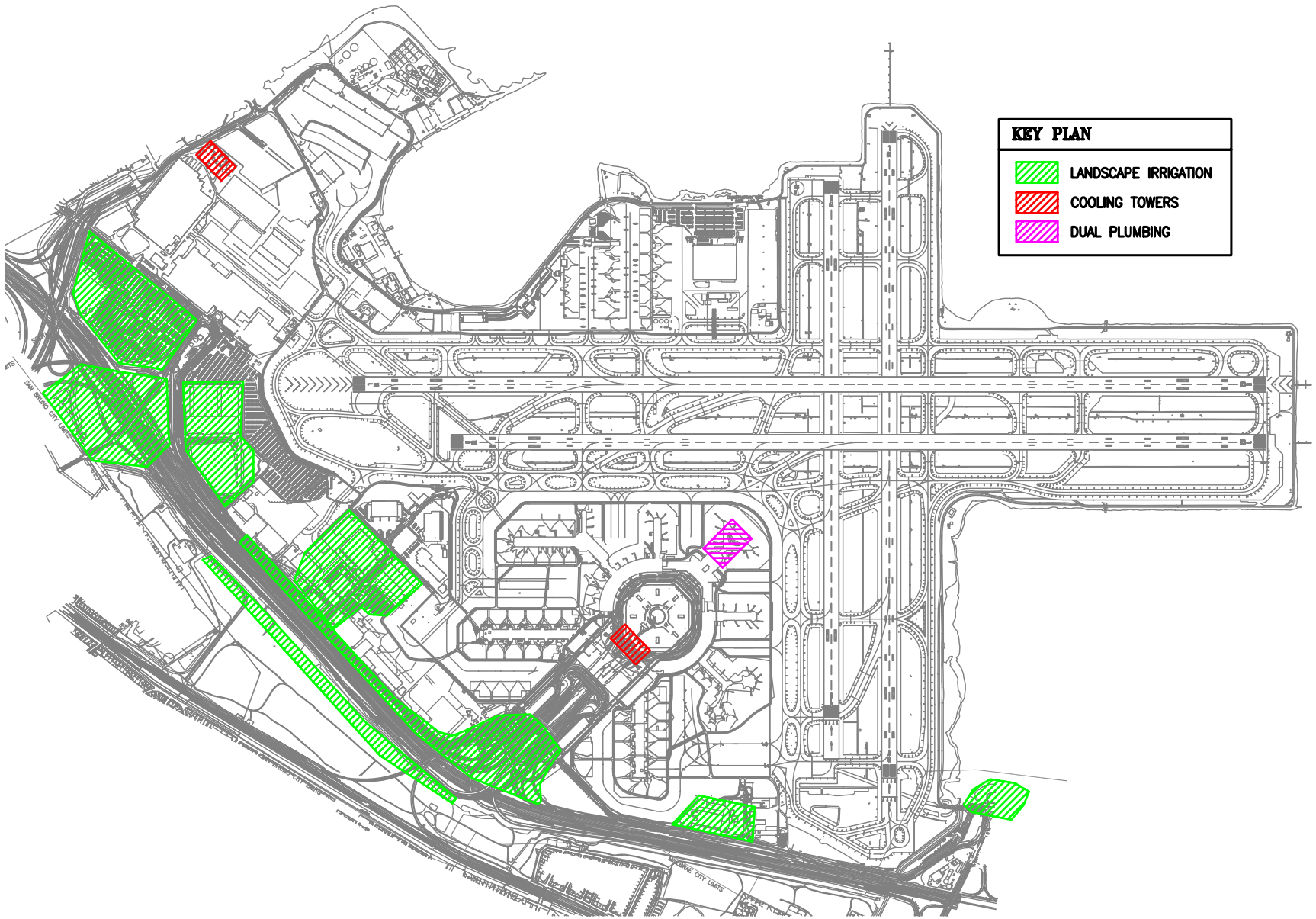
Table ES-2
Total and Annual Scenario Costs

Item	Scenario 1	Scenario 2	Scenario 3	Scenario 4
Total Program Cost	$17,625,000	$18,532,500	$16,582,000	$17,492,000
Total Annual O&M	$595,000	$625,000	$660,000	$675,000
Annualized Capital Cost	$1,189,000	$1,250,000	$1,118,000	$1,180,000
Net Annualized Cost	$1,784,000	$1,875,000	$1,778,000	$1,855,000

Since the net annual costs of each scenario are within a 4% range, other factors had to be evaluated to determine the best alternative. Scenarios 3 and 4 (decentralized treatment) offer more flexibility in phasing and implementation of the project. A tertiary treatment facility and distribution system can be constructed, independent of advanced treatment, to provide Title 22 water to the end user. Scenarios 1 and 2 (centralized treatment), on the other hand, would require tertiary and advanced distribution systems to be installed simultaneously. In the event that an advanced treatment facility is not constructed, all additional piping costs for the advanced treatment distribution system would be wasted.

A decentralized irrigation supply (scenarios 2 and 3) is desirable because of ease of control and ability to accommodate future increases in irrigation demands. A centralized irrigation supply (scenario 1 and 4) would require a large pump station capable of delivering water to far reaching locations at working pressures of 60-70 psi. More supply points and smaller pumps would make the irrigation supply system more robust and manageable.

Given the analysis, scenario 4 is the recommended alternative. The attached figures (ES.2, ES.3, ES.4, and ES.5) show the proposed distribution system, overall schematic flow diagram, and site layouts at the MLTP and in Lot C. Preliminary design criteria for scenario 4 for the treatment and distribution systems are included in Section 8.0 of the AAR.



SFO

San Francisco International Airport

Key Map

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PROJECT TITLE: AIRPORT RECLAIMED WATER – ALTERNATIVE ANALYSIS REPORT

SHEET TITLE: RECYCLED WATER END USERS FIGURE ES.1

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APPROVED: PRINCIPAL

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DRAWN BY: EL

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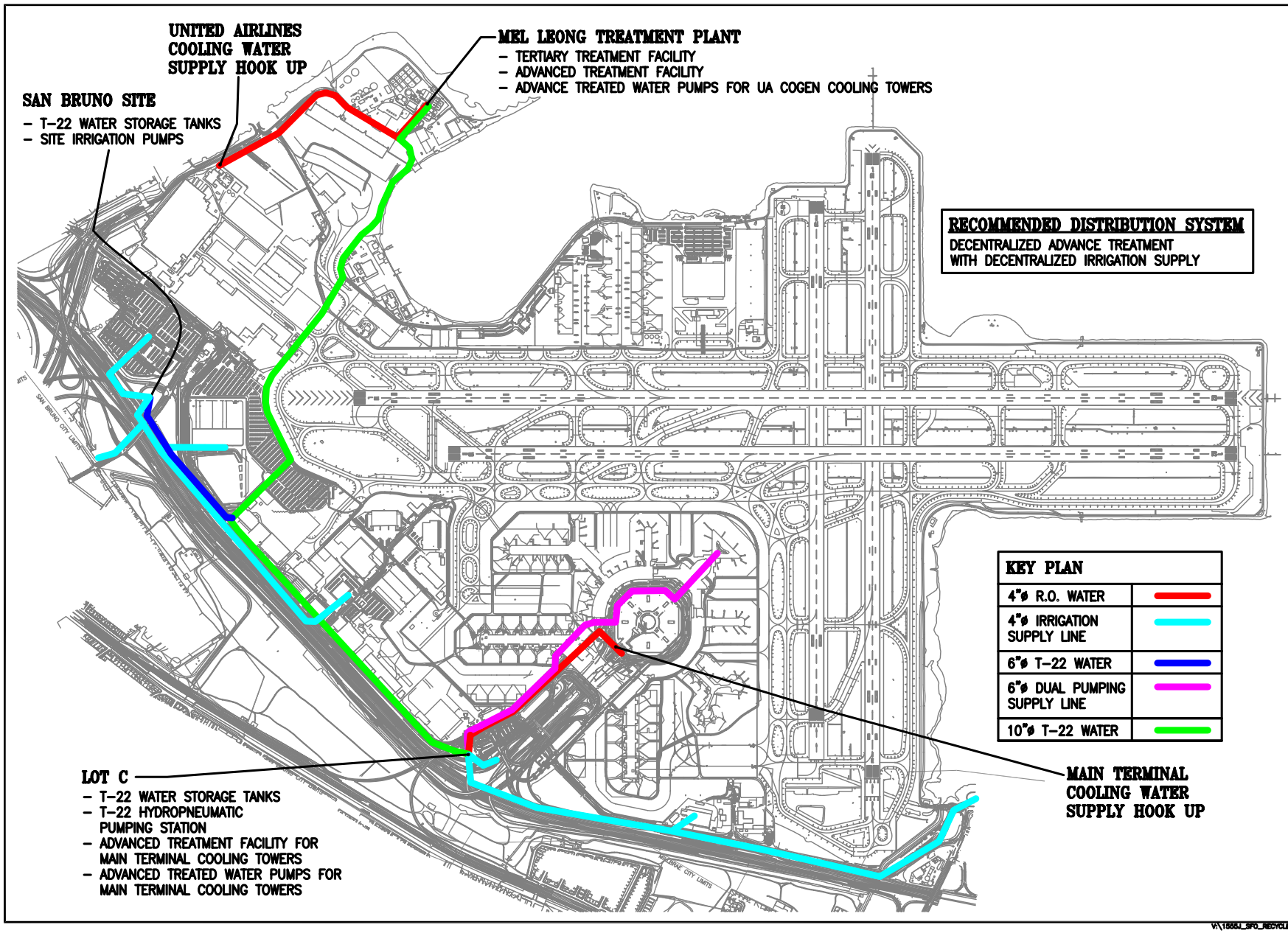

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PROJECT TITLE: AIRPORT RECLAIMED WATER – ALTERNATIVE ANALYSIS REPORT

SHEET TITLE: RECOMMENDED DISTRIBUTION SYSTEM FIGURE ES.2

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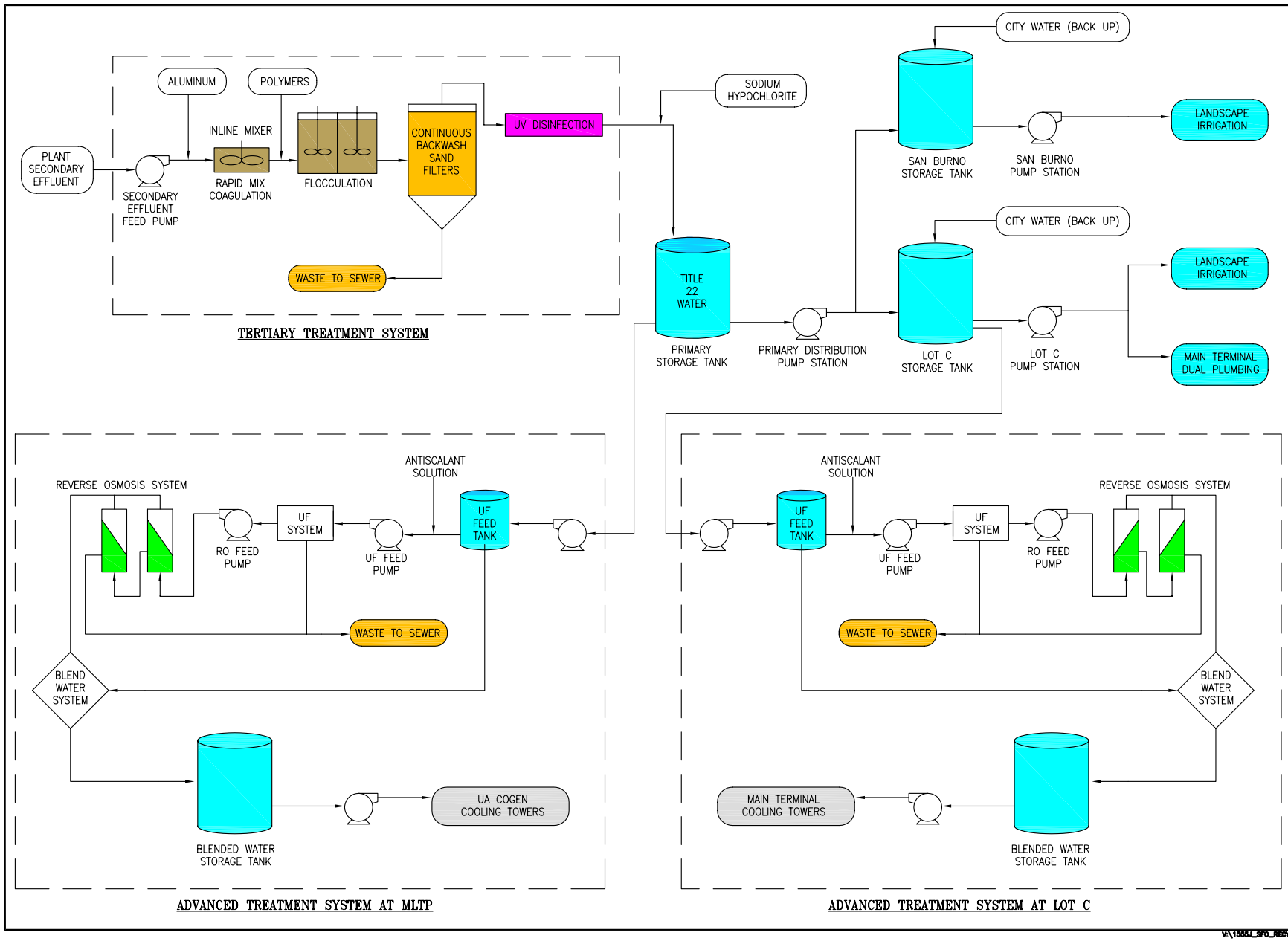

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SHEET TITLE: RECOMMENDED PROCESS FLOW DIAGRAM FIGURE ES.3

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SITE MAP

TERTIARY TREATMENT FACILITY IN MLTP

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SHEET TITLE RECOMMENDED SITE FIGURE ES.4

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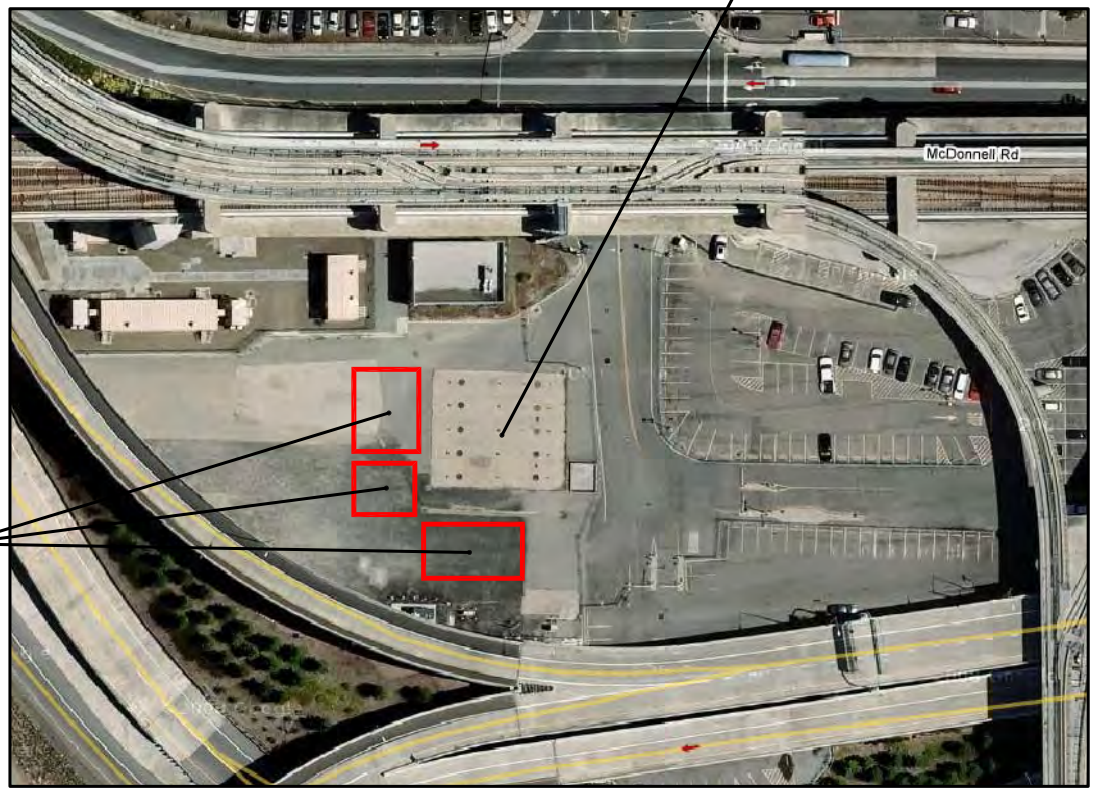
SITE MAP



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SHEET TITLE: RECOMMENDED SITE FIGURE ES.5

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2.0 Recycled Water Demand Estimates

2.1 Introduction

SFIA's preliminary planning report proposed the following end users as part of the SFIA Recycled Water Project:

- Landscape irrigation
- Cooling towers
- Dual plumbed facilities
- Runway median irrigation
- Groundwater recharge

During evaluation, the runway median irrigation and groundwater recharge uses were omitted from the list. Irrigating the runway medians would facilitate plant growth and attract birds and insects, creating a hazardous environment for potential bird strikes. Groundwater recharge was eliminated because of the difficulties of obtaining a permit approval from the Department of Public Health.

Thus, the Recycled Water Project treatment process equipment will be sized to meet the demands for the following end users: landscape irrigation, dual plumbed facilities, and cooling towers.

2.2 Demand Estimates from Preliminary Planning

Preliminary planning phase has estimated the following demands for the end users.

Table 2-1
Summary of Demand Estimates from Preliminary Planning

	Annual Avg (MGD)	Summer (May-Oct) Avg (MGD)	Winter (Nov-Apr) Avg (MGD)
Landscape Irrigation	0.10	0.15	0.04
Dual Plumbing	0.16	0.16	0.16
Cooling Towers	0.33	0.33	0.33
Total	**0.59**	**0.64**	**0.53**

2.3 Review and Refinement of Estimated Demands

2.3.1 Irrigation Demands

The irrigation demands were estimated using water meter readings and by interviewing Airport staff. While the majority of the water meters are dedicated to the irrigation system, some irrigated areas are extensions of building water meters; therefore, a

component of error exists in the water meter data for irrigation. This data, confirmed by Airport landscapers, is the basis for future irrigation demands. See Figure 2.1 for the graphical representation of the irrigation demands per the Airport landscaping staff. Future actual demands, however, could exceed these initial estimates. In supplying recycled water for irrigation in lieu of domestic water, the Airport staff may be more inclined to irrigate the ground more often.

2.3.2 Dual Plumbing Demands

The estimated dual plumbing demands are based on terminal "population" data. Passenger enplane/deplane data was used to determine the total amount of people in the terminals per month for the past five years. An additional 5,000 people per day were added to the data to account for general public, custodians, airline employees, tenants, etc. It was assumed that each person flushed at least once a day and each flush is 1.6 gallons, (typical of standard water closets). This figure is conservative because not all dual plumbed facilities will be 1.6 gallon water closets (i.e. a percentage of men will only use urinals which require 0.8 gallons per flush).

Given the analysis, the results are as follows:

Table 2-2
2004-2008 Terminal Population Flush Data

	Population (Million)	Total Water Flushed (MGY)	Estimated Flush Demand (MGD)
2004	33.98	54.37	0.148
2005	34.62	55.40	0.151
2006	34.90	55.85	0.153
2007	37.14	59.42	0.163
2008	38.89	62.22	0.170

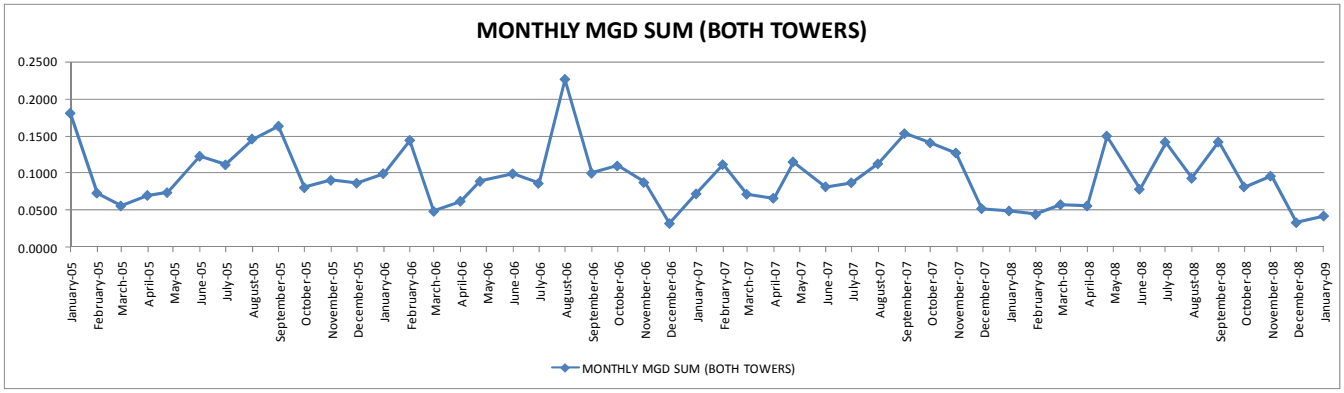
The annual increase in flush demand is consistent with the annual growth characteristics of the Airport. Looking forward, the Airport is expecting a slight decrease in growth for fiscal year 2009 in the range of 1 to 3%. In fiscal year 2010, a year-over-year decline in the range of 2 to 4% is projected. Starting in fiscal year 2011, an average annual growth rate in the range of 2 to 2.5% is projected through fiscal year 2015.

2.3.3 Central Plant Cooling Tower Demands

The cooling tower demands were estimated using the water meter readings for each cooling tower located at the SFIA's Central Plant. The dedicated water meters only supply cooling towers. To obtain an estimate of the peak flow and average flow demands between the cooling towers, the Airport analyzed the monthly water meter readings from January 2005 through December 2008. The peak "dry weather months" demand of a cooling tower occurs between May and October. Given the four year historical data, the average demand was calculated to be 0.1 MGD as shown in Fig 2.2.

Figure 2-2

Four Year Historical Data of the Central Plant Cooling Tower Demands



2.3.4 United Airlines Cogeneration Facility Cooling Tower Demands

The United Airline Cogeneration facility cooling tower demands were determined via staff interviews and a site visit. The staff stated that the demand ranged from 0.135 - 0.145 MGD for the summer of 2008.

2.3.5 Mel Leong Treatment Plant Demands

An additional 0.1 MGD was added to the total annual demand to replace any recycled (No. 3) water that MLTP may currently use with the existing recycled water facility. Currently, MTLP uses No. 3 water for local irrigation and equipment washing.

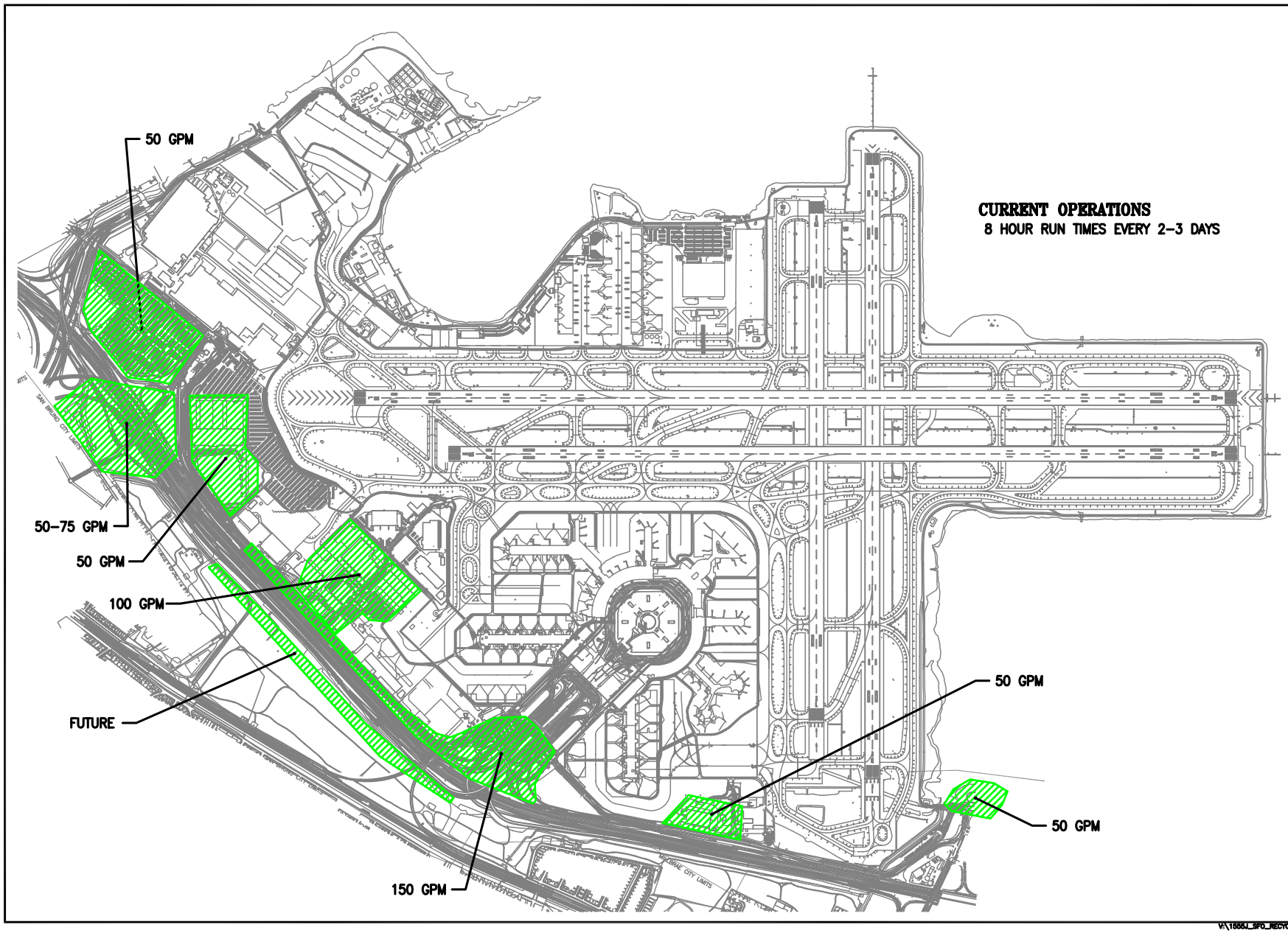
2.4 Summary of Refined Demand Estimates

Refined demand estimates were calculated for each of the end users and are listed in Table 2-3.

Table 2-3

Refined Demand Estimates

	Annual Avg (MGD)	Summer (May-Oct) Avg (MGD)	Winter (Nov-Apr) Avg (MGD)
Landscape Irrigation	0.10	0.16	0.04
Dual Plumbing	0.17	0.17	0.17
Cooling Towers - Airport	0.13	0.15	0.11
Cooling Towers - Cogen	0.15	0.15	0.15
MLTP	0.10	0.10	0.10
Total	**0.65**	**0.73**	**0.57**



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SHEET TITLE: LANSCAPE IRRIGATION FIGURE 2.1

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3.0 Alternatives Analysis Approach

3.1 Introduction

Recycled water can be produced at several locations using different treatment processes, and water can be distributed to end users using many routes. To identify the best combination of choices, a systematic and objective approach was used to analyze and rank the alternatives. This section describes the approach used to identify alternatives with the most promise for achieving the objectives of the SFIA Recycled Water Project.

The alternatives analysis process includes:

- Development of statement of objectives.
- Initial screening of alternatives using boundary conditions.
- Establishment of evaluation criteria to measure relative benefits of alternatives
- Assessment of net benefits of alternatives
- Assessment of costs of alternatives
- Assessment of benefits-to-cost ratios of alternatives
- Identification of the programmatic alternative that offers the best value

Each step was completed for identifying alternative treatment technologies and site for the treatment facilities. Each treatment technology was evaluated in relation to the identified site for treatment to establish the recommended program.

3.2 Statement of objectives

Objectives are broad statement of the goals that must be achieved for the program to be successful. Generally, a statement of objectives should be specific, measurable, attainable, realistic, and time-bound. The following objectives of the RWP were established during meeting with SFIA staff;

- Recycled water will reduce reliance on water supplied from SFPUC and diversify SFIA overall water supply portfolio by providing a new drought –proof, sustained and reliable water supply.

- Recycled water facilities will be:
  - Operator friendly and functional
  - Cost effective

- The Recycled Water Program will foster credibility and confidence with public through expedited implementation

3.3 Boundary conditions

Boundary conditions were established to aid with the initial screening of alternatives. Boundary conditions are the minimum standards that must be met for an alternative to be considered feasible. Alternatives that do not meet the boundary conditions were removed from the list of viable alternatives.

3.3.1 Boundary Conditions for Treatment Facility Sites

Boundary conditions for treatment facility sites include:

- Space available for the treatment alternative being considered must be adequate for normal operation and maintenance, readily accessible to construct the facility and for delivery of chemicals upon completion of the project.
- SFIA must have permitting authority or be able to obtain permits to construct the facility and regulatory approval to use the site(s).
- The site(s) must have the appropriate land-use designation(s)/ zoning district(s).
- The site(s) must not have significant geological or contamination hazards that would preclude construction of treatment facilities within the established schedule.
- The site(s) must not have cultural or environmental resources that would preclude construction of treatment facilities within the established schedule.

3.3.2 Boundary Conditions for Treatment Processes

Boundary conditions for the treatment process are:

- Treatment processes must be capable of meeting Title 22 standards for disinfected tertiary recycled.
- Treatment processes must be capable of achieving other water quality criteria required by the end uses that may go beyond the requirements of Title 22.
- Treatment processes must fit within the selected area.

3.4 Evaluation Criteria for Measuring Benefits

The ability of alternatives to achieve the objective of the program can be measured using evaluation criteria. Performance measures were established for each criterion to provide an objective means for quantifying the benefits of each alternative. Benefits are a

measure of the advantages offered by an alternative that are difficult to measure in the terms of cost.

1. **Implementability:** This criterion measures the feasibility of successfully constructing and operating an alternative. Items that may impact the feasibility of implementing an alternative that are beyond the items listed in other criteria include property acquisition, zoning changes, and approval by other government agencies.

2. **Reliability:** This criterion measures the ability of an alternative to consistently meet water quality and supply objectives without disruption. Alternative that can handle fluctuations in the quality of feedwater to the treatment process are preferable. In addition, alternatives that have a proven track record and have higher levels of redundancy are considered more desirable.

3. **Ease of Operations and maintenance:** This criterion measures the amount of labor required to attain water quality and supply objectives. Alternatives that require less operator attention through automation or robustness of the system are preferable. In addition alternatives that less complex and have fewer routine maintenance requirements are preferable.

4. **Impacts to Cultural and environment resources:** This criterion measures the amount of cultural and environmental resources that may be affected by construction and operation of the facilities. Fewer impacts preferable.

5. **Geological contamination hazards:** This criterion measures the relative risk alternatives have from geological and contamination hazards. Geological hazards include the liquefaction potential during seismic events landslides, and tsunamis. Contamination hazards include the potential for encountering known soil or water contamination during construction of an alternative.

6. **Staff Training Requirement:** This criterion measures the relative amount of training required to operate and maintain an alternative. Alternatives with fewer training requirements are preferable.

7. **Implementation Schedule:** This criterion measures the duration required to implement an alternative. Alternative with more complex construction and mitigation requirements will take longer to implement and, therefore, less desirable.

Once the criteria were established, team members were asked to weight the relative importance of the criteria. Each item was weighed using the following scale:

1. Trivial
2. Not very important
3. Important
4. Very important
5. Critical

As a result, some criteria were more heavily weighted in the decision model to reflect their relative importance.

3.5 Cost Estimates

Planning-level cost estimates were prepared to aid the analysis of alternatives. The estimates include:

- Capital Costs, which include construction costs and non-construction costs (i.e., planning, permitting, design, construction management, legal counseling, and administration).
- Operation and maintenance cost.
- Net present value and annualized costs.

The cost estimates were prepared using industry-standard cost estimating practices, references, costs of similar projects, and material quotes from vendors. The costs presented herein are considered a *Class 4 estimate*, as defined by the Association for the Advancement of Cost Engineering International for construction cost estimating. Class 4 cost estimates are generally prepared during preliminary planning to form the basis for the project authorization and/or funding. The typical expected accuracy range for this class of estimate is -15 to -30 on the low side and +20 to +50 on the high side.

The opinions of cost shown and any resulting conclusion on project financial or economic feasibility or funding requirements have been prepared for guidance in project evaluation and implementation from the information available at the time the opinion was prepared. The final costs of the project and resulting feasibility will depend on actual labor and material costs, competitive market conditions, actual sites condition, final project scope, implementation schedule, continuity of personnel and engineering and other variable factors. As a result, the project costs may vary from the opinions of cost presented herein. Given these factors, project feasibility, benefits/cost ratios, risk, and funding needs must be carefully reviewed prior to making specific financial decision or establishing project budgets to help ensure proper project evaluation and adequate funding.

3.6 Benefit-to-Cost Ratios

A benefit-to-cost ratio is simply the net benefit score of an alternative divided by its net present value. Alternatives with higher benefit-to-cost ratios are considered more desirable because these alternatives achieve the technical objectives at the most reasonable cost. Comparing the benefit-to-cost ratios for each alternative aids in the decision-making process. (See section 3.4 for the measuring criteria to develop a benefit-to-cost ratio)

3.7 Conclusions

This process was followed to arrive at all conclusions and recommendations in the report.

4.0 Evaluation of Treatment Technologies

4.1 Introduction

A number of treatment processes are available to produce recycled water from secondary effluent or raw sewage pumped from local sewers. The alternatives were screened to identify the most promising technologies in a multi-step process. First, the alternatives were screened using a qualitative process that retained the most promising technologies for further evaluation. The remaining alternatives were then compared in terms of net benefits and costs to identify the alternative that offers the best value.

4.2 Water Quality Requirements

The treatment technologies presented herein for producing recycled water have been approved under Title 22 CCR, Division 4, Environmental Health, Chapter 3, Recycling Criteria (adopted December 2000) (Title 22). The technologies are described in a September 2008 report Treatment Technology Report for Recycled Water (DHS 2008). Recycled water used for irrigating landscaping (Title 22, Section 60304), filling unrestricted recreational impounds (Title 22, Section 60305), cooling (Title 22, Section 60306), and other purposes such as flushing toilet and urinals (Title 22, Section 60307), must be treated to "disinfected tertiary recycled water" standards, as defined in Title 22, using approved treatment methods. Some of the pertinent details of the requirements of Title 22 are described below.

Recycled water that may be used for United Airline Co-generation plant and Airport Central cooling towers will require advanced treatment beyond Title 22 standards to further reduce the Total Suspended Solids (TSS) and level of conductivity to an acceptable level to prevent scaling and corrosion in cooling tower piping. The advanced treatment processes described herein are based on similar projects in many parts of California where recycled water is used for similar applications.

4.3 Overview of Treatment Technologies

Descriptions of the treatment processes, schematic process diagrams, and preliminary design criteria are provided below. The treatment technologies are grouped into the following categories:

1. Technologies for producing "tertiary disinfected recycled water" from secondary effluent:
   - Pretreatment
   - Filtration
     - Non-membrane filtration
     - Membrane filtration
   - Disinfection
2. Technologies for producing "tertiary disinfected recycled water" from raw sewage:
   - Membrane Bio-Reactor (MBR)
3. Advanced treatment to produce water suitable for use by Airport Central and United Airline Cooling Towers: • Reverse osmosis and advanced oxidation

4.4 Pretreatment Technologies

Pretreatment processes involve coagulation and flocculation of particles suspended in the feed water to form larger particles (flocs) that are easier to remove in clarification and filtration processes. Pretreatment is typically used ahead of non-membrane filtration processes, but it can be used before membrane filtration too. Title 22 contains the following pretreatment requirements for recycled water that is used for irrigation which is produced using non-membrane filtration:

> Recycled water used for the surface irrigation of the following shall be a disinfected tertiary recycled water, except that for filtration pursuant to Section 60301.320(a) coagulation need not be used as part of the treatment process provided that the filter effluent turbidity does not exceed 2 NTU (nephelometric turbidity unit), the turbidity of the influent to the filters is continuously measured, the influent turbidity does not exceed 5 NTU for more than 15 minutes and never exceeds 10 NTU, and that there is the capability to automatically activate chemical addition or divert the wastewater should the filter influent turbidity exceed 5 NTU for more than 15 minutes:
>
> a) Food crops, including all edible root crops, where the recycled water comes into contact with the edible portion of the crop,
>
> b) Parks and playgrounds,
>
> c) School yards,
>
> d) Residential landscaping,
>
> e) Unrestricted access golf courses, and
>
> f) Any other irrigation use not specified in this section and not prohibited by other sections of the California Code of Regulations.

It is likely that influent turbidity will sometimes exceed 5 NTU so at a minimum, pretreatment consisting of coagulation would be required to produce tertiary disinfected recycled water if non-membrane media filtration is used. Depending on the quality of feed water to the recycled water treatment facility and the selected filtration technology, additional pretreatment may be necessary to achieve compliance with water quality standards. Pretreatment is not required when membrane filtration processes are used, but it is often used when the feed water has very high turbidity.

Operating data obtained from daily grab samples of the final effluents of the Industrial and Sanitary treatment plants collected by the Mel Leong Treatment Plant (MLTP) lab between July, 2006 and February, 2008 indicates that the turbidity and concentration of total suspended solids (TSS) of the final effluent coming from the sanitary treatment plant can vary significantly. The 18 month operating data further indicate that the average turbidity of final effluent from the sanitary treatment plant exceeded 10 NTU, in September, 2006, and September through November, 2007 as shown on Figures 4-1.

Figures 4-1 through Figure 4-3 are plots showing variation in the monthly average turbidity for the sanitary, industrial and combined final effluents of the two treatment facilities. The average turbidity of the combined final effluent as shown in Figure 4-3 is less than 8 NTU for the most part of the year. However, the final effluent of the sanitary treatment plant shows significant

points of turbidity above 10 NTU for the periods the data was collected which therefore impacts the turbidity of the combined final effluent a few months out of the year. Although some remedial action can be taken to lower the turbidity of the sanitary treatment plant, the treatment technology recommended to achieve Title 22 requirements in this alternative analysis report is based on the assumption that there is no improvement planned in conjunction with the recycled water facility.

Figure 4-1
Sanitary Wastewater Final Effluent Turbidity

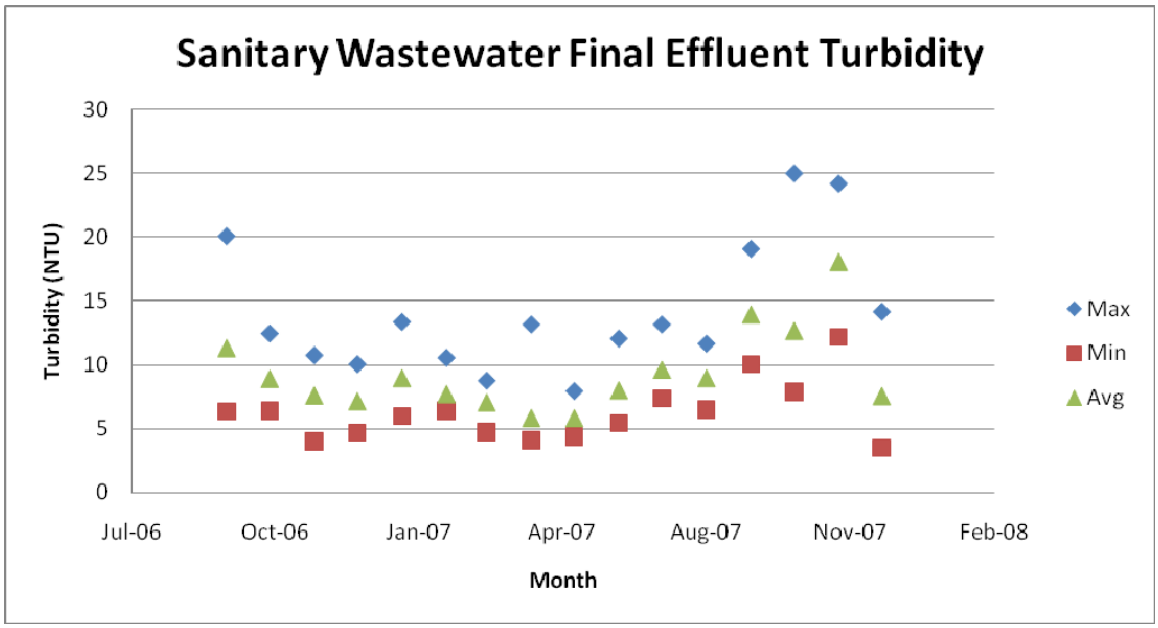


Figure 4-2
Industrial Wastewater Final Effluent Turbidity

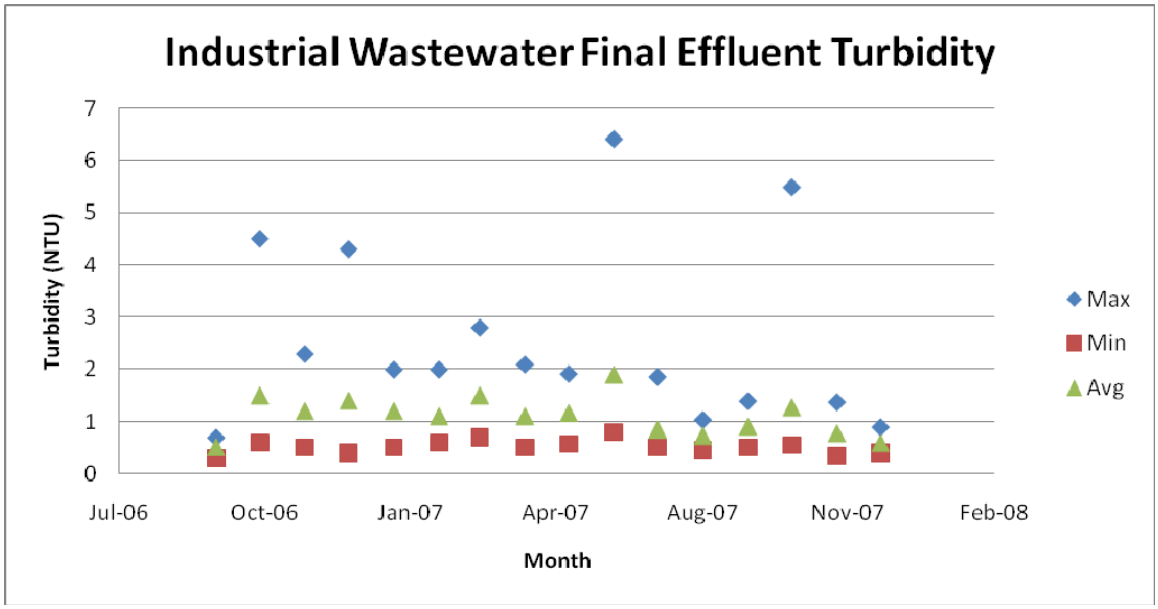
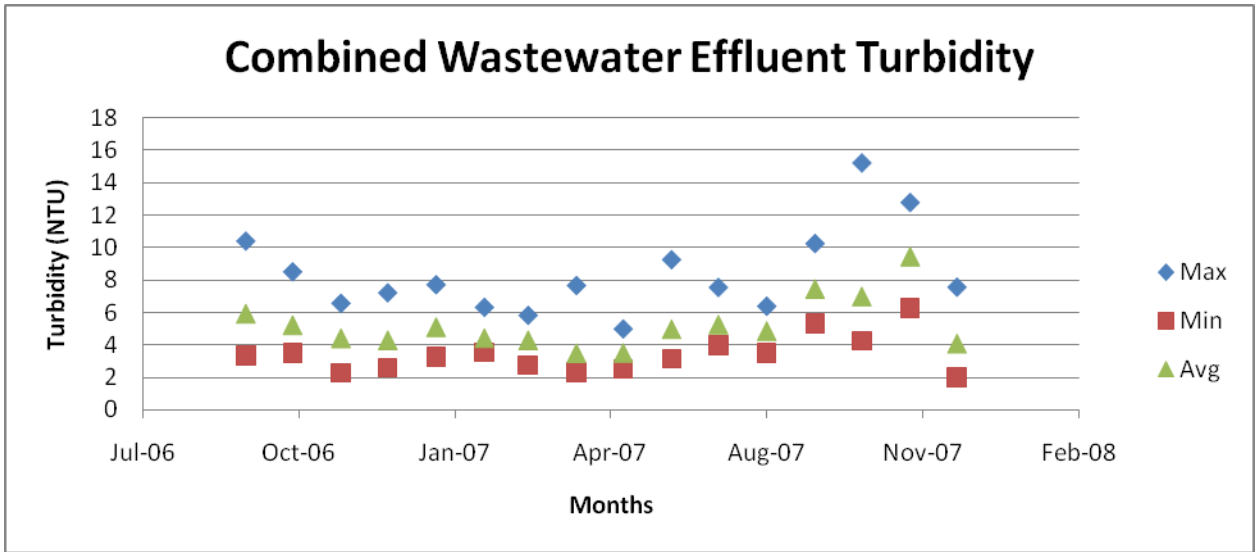


Figure 4-3
Combined Wastewater Final Effluent Turbidity



The following pretreatment processes, which are described in detail below, are commonly used:

- Rapid-mix coagulation before in-line filtration
- Rapid-mix coagulation and flocculation before direct filtration
- Rapid-mix coagulation, flocculation, and conventional clarification before filtration
- Rapid-mix coagulation, flocculation, and lamella plate/tube clarification before filtration
- I ligh-rate ballasted clarification (Actiflo®) before filtration

4.4.1 Rapid-mix Coagulation before In-line Filtration

This process would entail injecting a coagulant (such as alum or polyaluminum chloride) into feed water using an inline rapid-mixing device to destabilize charges on suspended particles, thereby allowing the particles to collide and agglomerate into larger particles. Capability to inject polymer solution downstream of the rapid mixer to help flocculate particles would also be provided. The mixed flow would then be conveyed to a downstream filtration process. This type of process is commonly called inline filtration. A schematic diagram of rapid-mix coagulation before inline filtration is shown in Figure 4-3.

Inline filtration does not include a clarification step between coagulation and filtration. Therefore, production of large flocs that can be removed easily in clarification is not necessary. Instead, inline filtration processes are usually designed to produce pinpoint-sized flocs that can be removed through filtration. Filterability improves as pin flocs undergo contact flocculation within the pore spaces of the filter media.

This process is best suited to high-quality feed water. If the feed water has high-turbidity (>10 NTU) or the feed water quality fluctuates rapidly, inline filtration ***may not be feasible***.

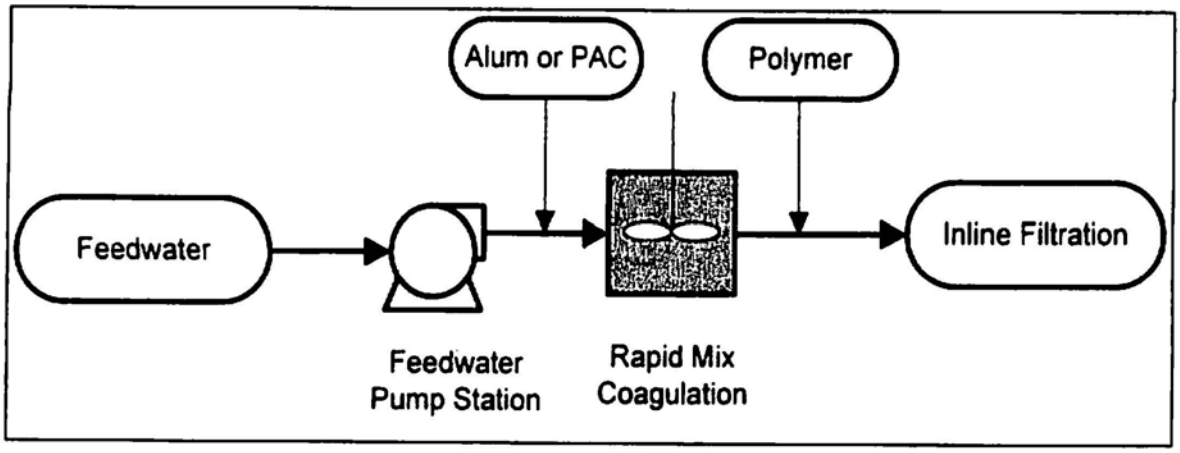
Advantages of this process include:

- Relatively low capital costs.
- Relatively low operation and maintenance costs.
- Low space requirements.
- Low chemical use.

Disadvantages of this process include:

- Relatively poor performance with high-turbidity feed water.
- Relatively poor capability to handle fluctuations in feed water quality.
- Requires a high level of automation and operator attention to react to variations in feed water quality.
- Only suitable for use with deep-bed granular media filtration.
- Does not decrease solids loading on filters.

Figure 4-4
Schematic Diagram of Rapid-mix Coagulation before Inline Filtration



4.4.2 Rapid-mix Coagulation and Flocculation before Direct Filtration

This process would be identical to inline filtration except that flocculation tanks would be inserted between the coagulation and filtration steps. Mechanical mixers would be installed in the flocculation tanks to promote collisions between polymer solution and coagulated solids in a controlled manner. This type of process is commonly called direct filtration. A schematic diagram of rapid-mix coagulation and flocculation before inline filtration is shown in Figure 4-5.

Flocculation using separate tanks may provide better results than inline filtration because the detention time can be increased, and the applied mixing energy can be controlled. Tests have shown that flocculation improves filtrate quality before breakthrough, shortens the initial improvement period of the filter cycle (i.e., the filter "ripening" period), and reduces the rate of headloss increase. However, flocculation can result in earlier breakthrough, which shortens filter cycles. Detention time in the flocculation tanks is typically between 10 and 20 minutes.

Direct filtration plants can be designed to bypass the flocculation step to provide additional operational flexibility. Therefore, performance of direct filtration is expected to be more robust

compared to inline filtration and the capability to handle fluctuations in feed water quality would improve.

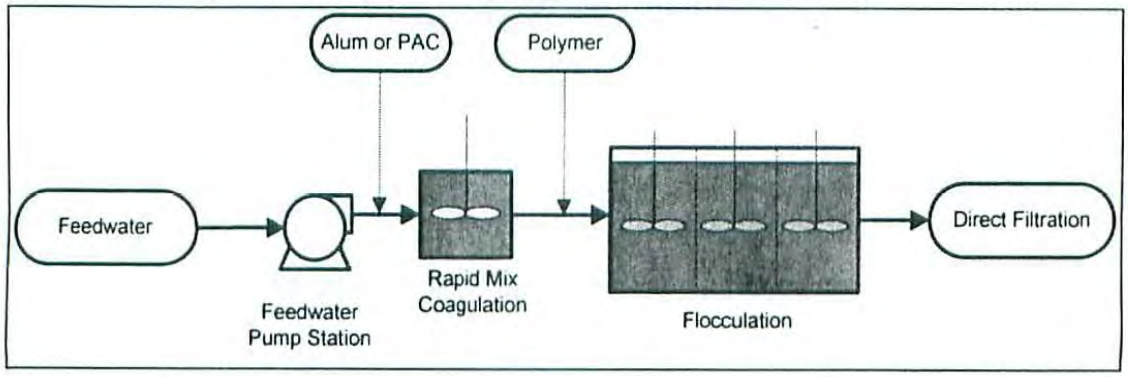
Conversely, this process would require more land space due to the addition of flocculation tanks. It would also be more expensive to construct compared to In-line Filtration.

A clarification step to remove flocculated solids before filtration would not be provided. Therefore, this process would not reduce solids loading on the filtration process.

Advantages of this process include:
- Moderate capital costs.
- Relatively low operation and maintenance costs.
- Moderate space requirement.
- Low chemical use.

Disadvantages of this process include:
- Relatively poor performance with high-turbidity feed water.
- Moderate capability to handle fluctuations in feed water quality.
- Requires a high level of automation and operator attention to react to variations in feed water quality.
- Only suitable for use with deep-bed granular media filtration.
- Does not decrease solids loading on filters.

Figure 4-5
Schematic Diagram of Rapid-mix Coagulation before Inline Filtration



4.4.3 Rapid-mix Coagulation, Flocculation, and Conventional Clarification

This process would be similar to the direct filtration process except that conventional circular or rectangular clarifiers would be provided after flocculation to remove settleable solids before they can reach the filtration process. Because solids loading on the filters would be reduced, this process would be more robust and reliable than the first two processes. However, this process would require additional land space for construction of clarifiers. It would also cost more to

construct than the first two processes. A schematic diagram of rapid-mix coagulation, flocculation, and conventional clarification is shown in Figure 4-6.

Surface loading on conventional clarifiers can be as high as 800 gallons per day per square foot (gpd/ft^2), which is much lower than other clarification technologies described below.

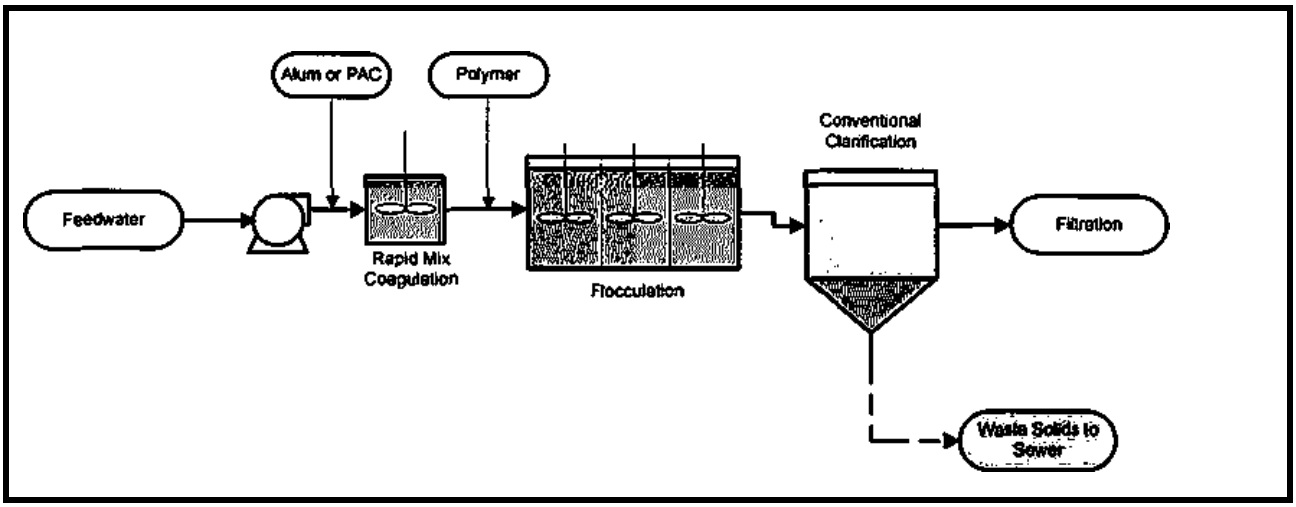
Advantages of this process include:

- Good performance with high-turbidity feed water.
- Good capability to handle fluctuations in feed water quality.
- Relatively low automation and operator attention required.
- Can be used with all approved filtration technologies.
- Decreases solids loading on filters.

Disadvantages of this process include:

- High capital costs.
- Moderately high operation and maintenance costs.
- High space requirement.
- Moderately high chemical use.

Figure 4-6
Schematic Diagram of Rapid-mix Coagulation before Inline Filtration, Flocculation, and Conventional Clarification before Filtration



4.4.4 Rapid-mix Coagulation, Flocculation, and Lamella Clarification

This option would entail rapid-mix coagulation and flocculation followed by lamella plate or tube clarifiers instead of conventional clarifiers. Lamella clarifiers use parallel inclined plates or tubes installed inside the clarifier that increase the effective surface area without increasing the size of the tank. Use of lamella clarifiers would reduce land space requirements while still providing the benefit of reducing solids loading on the filtration process. Lamella clarifiers can be coupled with proprietary treatment systems that incorporate rapid mixing and flocculation tanks in a complete pretreatment process. The treatment steps can also be kept separate in a

customized system. A schematic diagram of rapid-mix coagulation, flocculation, and lamella clarification is shown in Figure 4-8.

Figure 4-7
Lamella Plate Clarifier Parkson GEWE Plate Settler

Surface loading on lamella clarifiers can be as high as 6,000 gpd/ft^2, which is much higher than conventional clarifiers, but less than the Actiflo® and Densadeg® (Densadge is a product of *Infilco Degremont, not included in this report)* processes described below.

This option would use less land space than conventional clarifiers, but it would retain similar reliability and performance.

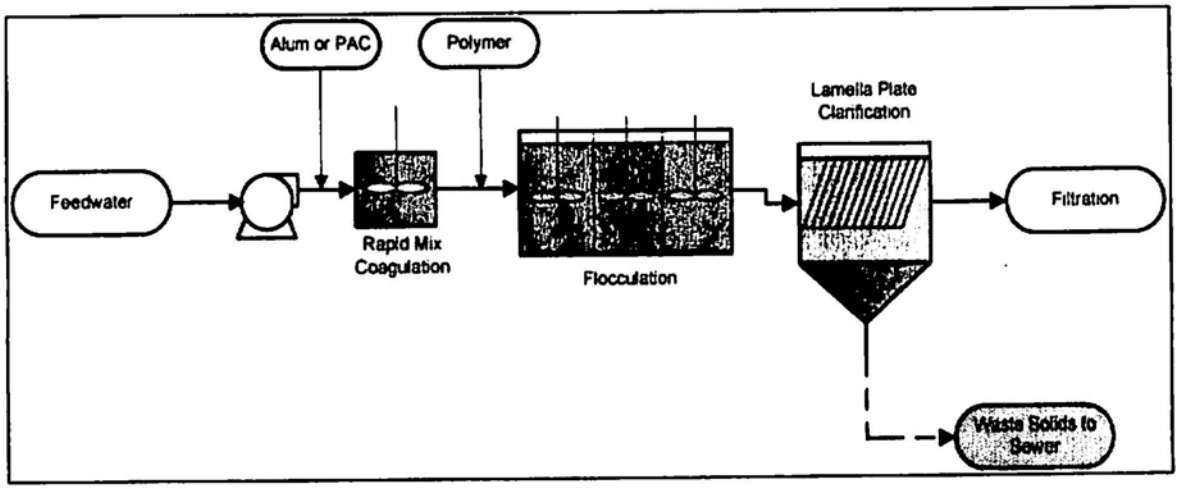
Advantages of this process include:

- Good performance with high-turbidity feed water.
- Good capability to handle fluctuations in feed water quality.
- Relatively low automation and operator attention required.
- Can be used with all approved filtration technologies.
- Decreases solids loading on filters.

Disadvantages of this process include:

- High capital costs.
- Moderately high operation and maintenance costs.
- Moderately high space requirement.
- Moderately high chemical use.

Figure 4-8
Schematic Diagram of Rapid-mix Coagulation, Flocculation, and Lamella Clarification before Filtration



4.4.5 High-rate Ballasted Clarification (Actiflo®)

This process would use the Actiflo® high-rate ballasted clarification process marketed by Kruger, Inc. ActiFlo® uses a three-step process that includes coagulation, flocculation, and clarification using inclined lamella settling tubes, as shown on Figure 4-9. In addition, Actiflo® uses microsand that is injected downstream of the coagulation tank to act as a nucleus for agglomerating solids into flocs. The relatively dense particles, which are made up of sand, solids, coagulant, and polymer, settle much more rapidly than flocs that do not include sand. As a result, much higher clarifier loading rates are possible. A schematic diagram of the Actiflo® process is shown in Figure 4-10.

The Actiflo® process consists of the following subsystems:

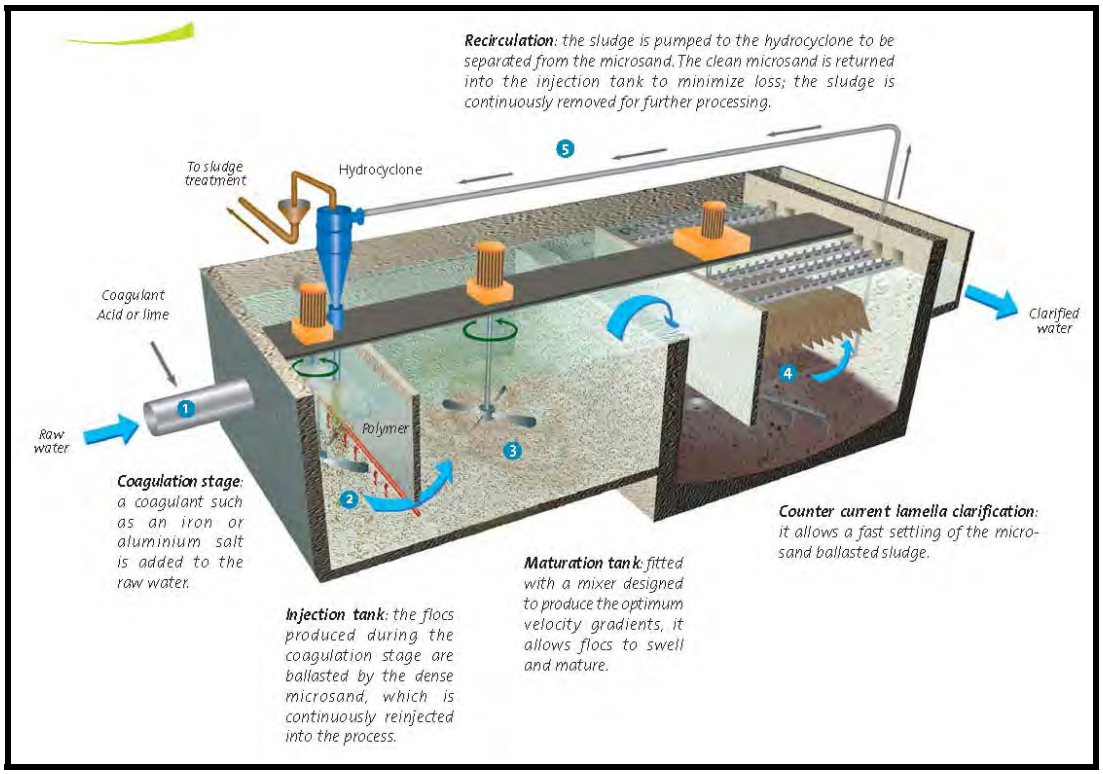
- Rapid-mix coagulation tank
- Sand and flocculation aid injection tank
- Maturation tank
- Settling tank
- Sand recirculation pump
- Hydrocyclones to separate sand and solids

The detention time in the coagulation and flocculation steps is typically about 6 to 10 minutes. A coagulation aid (such as alum) and a flocculation aid (such as polymer solution) are used to help form flocs that are separated from water in the clarifier zone. Settled flocs are pumped from the bottom of the clarifier through hydrocyclone separators to segregate relatively dense sand from the captured solids. The cleaned sand is recycled through the process, while the solids are removed from the process. The waste sludge typically has suspended solids concentrations ranging from 0.1 to 0.2 percent solids.

Surface loading rates for the Actiflo® process can be as high as 36,000 gpd/ft^2, which significantly reduces the required surface area compared to other technologies.

Actiflo® is advantageous compared to conventional pretreatment processes because it requires much less space. It is also capable of handling large fluctuations in feed water quality. On the other hand, Actiflo® is a proprietary technology that is more complicated to operate than conventional processes. In addition, it uses relatively high amounts of energy and conditioning chemicals. Therefore, Actiflo® is usually limited to situations where limited space is available and/or the quality of the feed water is poor.

Figure 4-9
Configuration of Actiflo® High-rate Ballasted Clarification Process



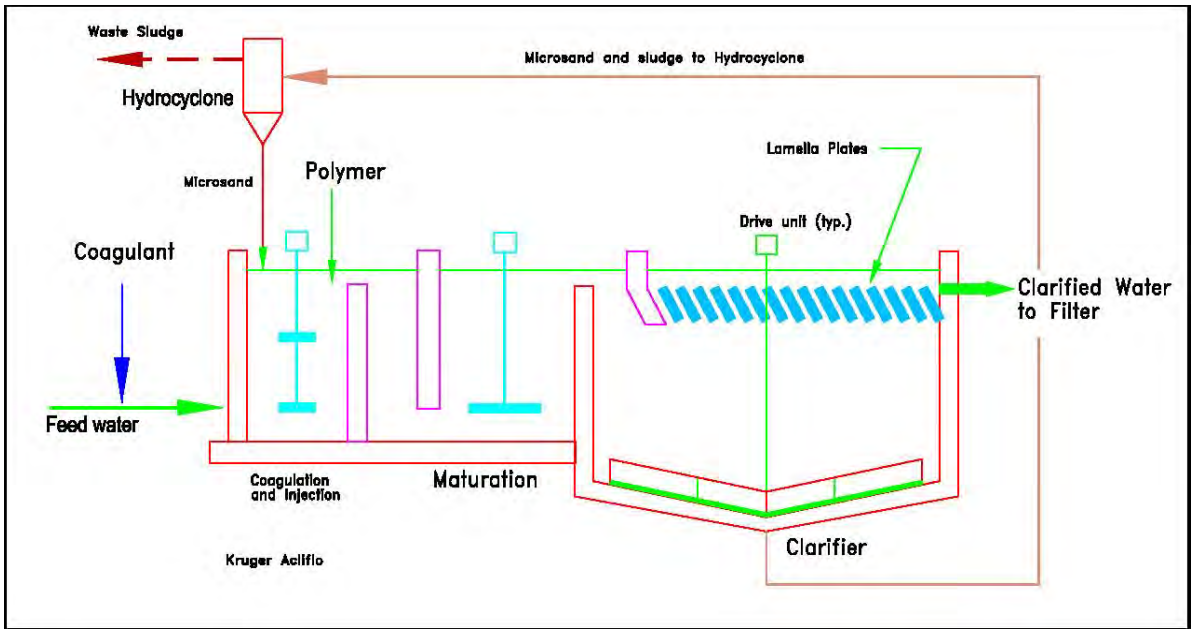
Advantages of this process include:

- Excellent performance with high-turbidity feed water.
- Excellent capability to handle fluctuations in feed water quality.
- Moderate automation and operator attention required.
- Can be used with all approved filtration technologies.
- Decreases solids loading on filters.
- Rapid startup period.

Disadvantages of this process include:

- High capital costs.
- High operation and maintenance costs.
- Moderate space requirement.
- High chemical use.

Figure 4-10
Schematic Diagram of High-rate Ballasted Clarification (Actiflo®)



4.4.6 Summary of Pretreatment Alternatives

Table 4-1 presents a summary of the pretreatment alternatives. It is recommended that all of the alternatives except Alternative 3 (Rapid-mix Coagulation, Flocculation, and Conventional Clarification) be retained for further evaluation. Alternative 3 is less desirable because it requires much more land space, and construction costs are expected to be relatively high. Moreover, this alternative does not provide any treatment benefits compared to options that use much smaller lamella clarifiers.

It is recommended that alternatives that involve inline and direct filtration (i.e., no clarification ahead of filtration) be retained for further evaluation, but these alternatives should only be considered as pretreatment ahead of granular media filters with bed depths of at least 80 inches (2 meters). The alternatives that do not include clarification are not suitable for pretreatment ahead of granular media filters with shallower bed depths or cloth-media filters because the combined effluent turbidity is beyond the limits of the feedwater quality specified by the manufacturers of the system.

Alternatives 4 and 5, which all include clarification ahead of filtration, would provide better pretreatment for all granular-media and cloth-media filtration processes, however we are reasonably confident that the need for this type of system may not be required because the average quality of the combined effluent of the SFO treatment plants can be treated using Alternatives 1 and 2 to achieve Title 22 water quality requirements.

In addition to the use of flocculation tank of alternative 2, we are investigating the possibility of using a fine screen strainer ahead of the rapid mix coagulation process to eliminate the need of

using a flocculation tank. The installed cost of a fine screen strainer (which requires significantly less space) is considerably lower than a 4-stage flocculator that may be needed for use in this project. Likewise the cost of energy used during flushing to backwash the strainer is considerably lower than the chemical and energy cost of the flocculator.

As noted previously, the pretreatment processes listed here are not required ahead of membrane filtration processes. However, membrane filtration would require that the influent flow be passed through fine screen strainers to prevent fouling of membranes. These requirements are discussed in more detail below.

Table 4-1
Comparison of Pretreatment Technologies

Pretreatment Technology	1	2	3	4	5
Description	**Rapid-mix Coagulation for Inline Filtration**	**Rapid-mix Coagulation and Flocculation for Direct Filtration**	**Rapid-mix Coagulation, Flocculation, and Conventional Clarification**	**Rapid-mix Coagulation, Flocculation, and Lamella Clarification**	**High-rate Ballasted Clarification (Actiflo)**
Ability to Treat High-turbidity Water	▼	◙	▲	▲	▲▲
Ability to Handle Water Quality Fluctuations	▼	◙	▲	▲	▲▲
Level of Automation and amount of Operator Attention Required	▼▼	▼	▲	▲	◙
Decreased Loading on Filters	▼	◙	▲	▲	▲▲
Complexity	▲▲	◙	▼	◙	▼▼
Chemical Consumption	▲	▲	◙	◙	◙
Land Space Required	▲	◙	▼▼	◙	◙
Capital Costs	▲	◙	▼▼	▼	▼
O&M Cost	▲	▲	◙	◙	▼
Recommendation	Retained for further evaluation, but only as pretreatment for deep-bed granular media filtration	Retained for further evaluation, but only as pretreatment for deep-bed granular media filtration	Eliminate from further consideration due to space and cost issues. Lamella plate clarifiers are a better alternative.	Retained for further evaluation.	Retained for further evaluation.

Legend:

▲▲	Superior
▲	Good
◙	Average
▼	Fair
▼▼	Inferior

4.5 Filtration Technologies

Title 22 of the California Water Recycling Criteria (adopted December 2000) defines Disinfected Tertiary Recycled Water as a wastewater, which has been oxidized and meets the following:

A. Has been coagulated and passed through natural undisturbed soils or a bed of filter media pursuant to the following:*

1. At a rate that does not exceed gpm/ft² in mono, dual or mixed media gravity or pressure filtration systems, or does not exceed 2 gpm/ft² in traveling bridge automatic backwash filters; and

2. The turbidity does not exceed any of the following; a daily average of 2 NTU, 5 NTU more than 5% of the time within a 24-hour period, and 10 NTU at any time.

**Note: Coagulation may be waived if the filter effluent does not exceed 2 NTU, the filter influent is continuously measured, the filter influent turbidity does not exceed 5 NTU, and automatically activated chemical addition or diversion facilities are provided in the event filter effluent turbidity exceeds 5 NTU.*

OR

B. Has been passed through a micro, nano, or R.O. membrane following which the turbidity does not exceed any of the following: 0.2 NTU more than 5% of the time within a 24hour period and 0.5 NTU at any time.

AND

C. Has been disinfected by either:

1. A chlorine disinfection process that provides a CT of450 mg-min/l with a modal contact time of not less than 90 minutes based on peak dry weather flow, or

2. A disinfection process that, when combined with filtration, has been demonstrated to achieve 5-loginactivation of virus.

The DHS has established a list of filtration technologies that have been accepted for producing recycled water under Title 22, as shown in Table 4-2 (DHS September 2008). These technologies include granular media filters, cloth-media filters, membrane systems, and the Fuzzy Filter® sold by Schreiber, LLC.

Table 4-2
Accepted Filtration Technologies under Title 22

Category	Manufacturer and Product	Maximum Hydraulic Loading Rate, gpm/ft²	Example Installations in CA
Conventional Deep-bed (Granular Media)	Nonproprietary; Tetra Technology's Tetra-Denit.	5	Clear Creek; Monterey; Santa Barbara; Santa Rosa Laguna; Livermore; Glendale; Lake Arrowhead; Padre Dam; Scotts Valley
Upflow/Downflow Deep-bed Continuous Backwash (Granular Media)	Parkson's DynaSand; Waterlink's SuperSand; Andritz Ruthner's Hydrasand; Applied Process Technology's Centra-flo; Fluidyne's Fluidsand; Westech's Technasand; Davco's Astrasand; Ashbrook's Strata-Sand	5	Auburn; Carmel Valley Ranch; Coachella Valley; Escondido; EBMUD Richmond; Santa Rosa; Oakmont; Delta Diablo SD; Mt View SD; Napa; Dublin San Ramon; Camarillo; San

			Bernardino; Sausalito; Tiburon; Stockton; Las Galinas; Castanoa Ranch, San Mateo County; Oceanside
Shallow-bed Traveling Bridge (Granular Media)	Aqua-Aerobic's AquaABF; Infilco Degremont's AquaABF; US Filter Davco's Gravisand	2	Shasta Lake City; Stillwater; Folsom; Sacramento; Shasta Lake; Victor Valley Sepulveda; Los Angeles
Shallow Pulsed Bed (Granular Media)	US Filter Zimpro's HydroClear	5	Moulton Niguel Water District; San Luis Obispo; San Clemente; Rancho Murrieta; Fallbrook
Cloth Media	Aqua-Aerobic's AquaDisk and AquaDiamond; Kruger"s Hydrotech; Nordic Water Disc Filter; Parkson Dynadisc Siemens- 40X, and Five Star	6	Bodega Bay; Lodi; Huntington Beach: Penis; San Jacinto; Hopland; Hume Lake
Other Media	Schreibers Fuzzy Filter	30	Yountville
Membrane Filtration	Zenon/ZeeWeed US Filter/Memcor/CMF, CMF-S and Jet Tech products Memjet; Pall (Asashi)/ Microza™ GE Ionics (Norit)/XIGA™ Mitsubishi; Kubota; Koch/Puron; Huber Technologies Parkson/Dynalift Ashbrook Simon-Hartley IMAS DOW Ultrafiltration and several others		

4.5.1 Filterability of Combined Secondary Effluent from Mel Leong WPCP

The Sanitary wastewater treatment plant employs a Sequencing Batch Reactor (SBR) that was designed to treat up to 4.4 MGD in rain events. The quality of the turbidity of the final effluent of this plant shows significant variation ranging from below 5 NTU to a maximum of 25 NTU. However the inherent design of the SBR can be exploited to produce a much better effluent by optimizing the process and relaxing the Mayor's energy conservation directives to allow use of the blowers to achieve water quality objectives. On the other hand the Industrial wastewater treatment uses a conventional treatment process where the turbidity of the final effluent of less than 2 NTU throughout the year almost meets Title 22 requirement with the addition of a disinfection process only.

Since there were no pilot tests done to determine feasibility of using non-membrane filtration of the combined effluent to meet Title 22 requirements, we have planned to rent a mobile continuous-backwash upflow sand filter unit prior to moving forward on to the CER phase of this project. A Parkson DynaSand®-D2 Filtration system with an xx-inch bed of silica sand followed by a patented micro-sand will be scheduled for mid-June of this year to do pilot testing for a period of 4 weeks. Pilot test will be designed to simulate worst case condition to evaluate suitability of the granular media filtration for the SFO Recycle water project.

Typical wastewater filtrate qualities of the Parkson DynaSand®-D2 are:

- Turbidity 0.05 – 0.1 NTU
- BOD/TSS ‹ 3 mg/l
- 7 log of Cryptosporidium and Giardia removal

The system produces filtrates that are equivalent or better than low-pressure membrane Microfiltration process with slightly higher use of coagulant but with 1/8 of the power consumption. Since the D2 is also capable of removing metals, results of the pilot may help us determine if the quality of the filtrate water is good enough for use by UAL and Airport central cooling towers which may eliminate the need to have an RO facility thereby reducing the overall RWF cost. However since there are no published pilot test results that shows the D2's capability to reduce effluent conductivity, we do not put high hope of eliminating advanced treatment using RO filtration.

4.5.2 Non-membrane Filtration Technologies

The list of approved technologies includes 14 granular-media filters, three cloth-media filters, and one other media filter (the Fuzzy Filter) that have been accepted for producing recycled water through demonstrated ability to meet the water quality requirements of Title 22. Of the 17 granular-media filters, there are eight upflow, continuous backwash filters; one shallow pulsed-bed filter; three shallow-bed, traveling bridge filters; one deep-bed gravity mono-media (conventional) filter; and two downflow, continuous backwash filters.

4.5.2.1 Conventional Deep-bed Granular Media Filters

Conventional deep-bed filters generally consist of 36 to 72 inches of a single-, dual-, or tri-media (sand, anthracite, and/or garnet) beds supported by a gravel layer and an underdrain system, as shown on Figure 4-11. Secondary effluent flows downward through the media, and suspended particulate matter is retained within the media. At the end of a filter cycle, when limiting headloss or solids breakthrough begins or after a preset duration has lapsed, the filter is backwashed to clean the filter media using a portion of the water filtered previously. During a backwash cycle, flow through the filter is reversed, and air and/or water are introduced to scour the media. The material released by backwashing is collected in backwash troughs and conveyed back to an equalization tank. Typical backwash water consumption is 6 to 10 percent of filter throughput.

Filtered water is normally stored in a tank so it is available when needed for backwashing. Backwash water is sometimes drawn from the clear well of the filters (instead of a separate storage tank), but this practice can be infeasible if the backwash demand is greater than the production capacity of the filters that remain in service. Moreover, drawing backwash water directly from the clear well can disrupt downstream disinfection processes.

Equalization tanks typically are used to retain backwash flow to avoid overloading the filter plant or the sewer.

Capability to chlorinate the backwash water is normally provided to prevent fouling of the filter media and underdrains by biological growth.

The advantages of conventional deep-bed filtration are:

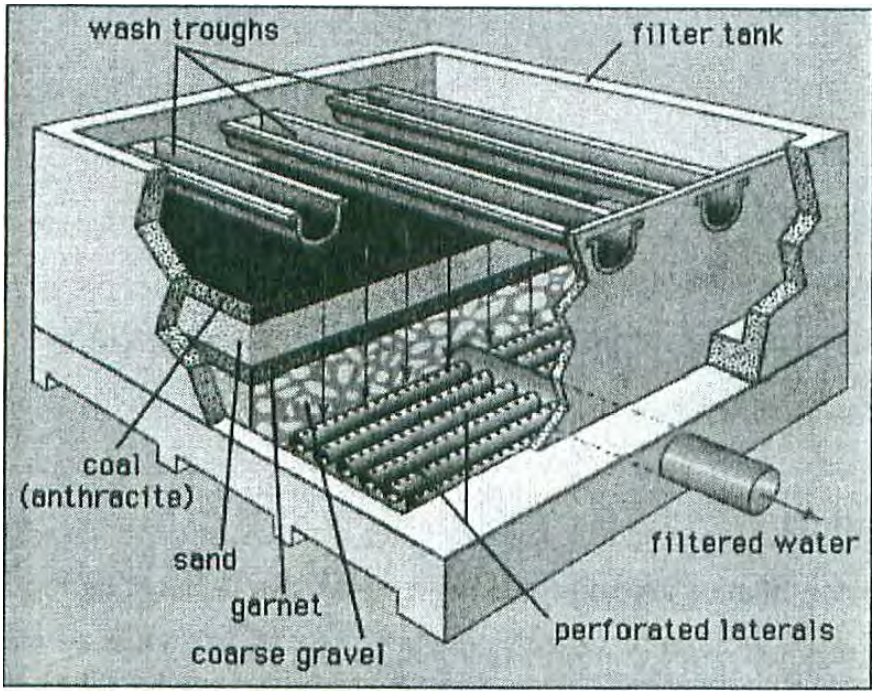
- Proven record of performance at municipal wastewater treatment plants in California and the United States.
- High solids loading capacity possible using deep beds.
- Able to handle process upsets and shock hydraulic loads.

The disadvantages of conventional deep-bed filtration are:

- Large footprint.
- Relatively high capital cost.
- Semi-continuous operation requires more redundancy.
- Backwash supply and waste equalization tanks required.
- High capacity backwash pumps required.
- Potential for dead zones and 'mud-ball' formation.
- Relatively complex design.

Because of the high cost, complexity, and large footprint required for conventional deep-bed filters, it is recommended that this technology be eliminated from further consideration. Other alternatives, such as continuous backwash sand filters, offer equivalent performance but are less complex and have smaller footprints.

Figure 4-11
Configuration of Convention Granular Media Filter



4.5.2.2 Upflow and Downflow Continuous-backwash Sand Filters

In upflow, deep-bed continuous backwash sand filters, wastewater is introduced at the bottom of the filter that flows upward through a downward-moving sand bed, as shown on Figure 4-12). The filtered effluent flows over a weir at the top of the tank and is discharged from the filter. When the sand and retained solids reach the bottom of the filter, they are conveyed upward through an airlift pump that provides turbulent mixing causing solids to be dislodged from the sand. The relatively dense sand is then recycled back to the top of the filter bed, while less dense solids are removed with waste flow from the airlift pump.

The clean sand is redistributed on top of the sand bed, allowing for continuous flow of filtrate and reject water; therefore, no downtime for backwashing is required.

Backwash consumption is typically 5 to 8 percent of filter throughput. Typical media depth is 40 inches (1 meter), but designs with bed depths up to 80 inches (2 meters) are available when needed to filter water that is more difficult to treat.

Downflow, deep-bed continuous backwash filters are similar to upflow filters except that the flow directions of water and sand are reversed. Upflow filters are generally preferred because they have a longer record of performance, they provide good results, and there are seven Title 22- accepted vendors of upflow continuous backwash filters, which results in strong cost competition.

The advantages of upflow deep-bed continuous backwash filtration are:

- Proven record of performance at municipal wastewater treatment plants in California and the United States.
- Continuous operation (no shutdown during backwash).
- Smaller footprint than conventional filters.
- No backwash supply tanks required.
- No backwash holding or equalization tanks required.
- No backwash pumps required.
- Low power requirements.
- Low operation and maintenance requirements.

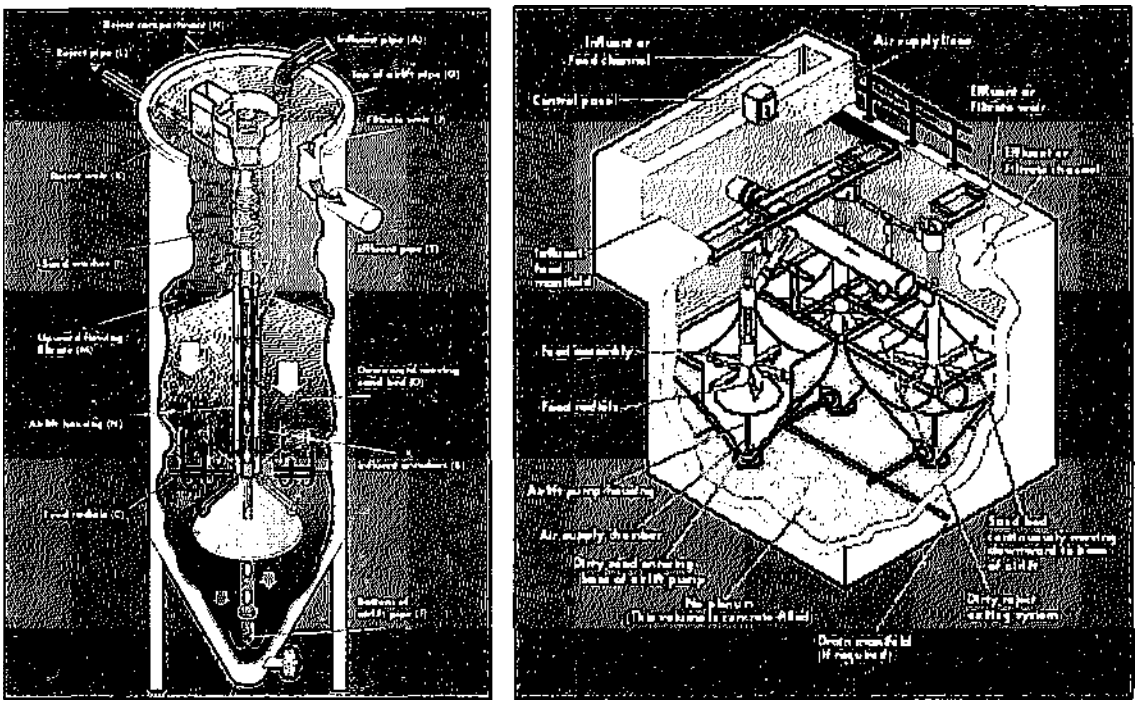
The disadvantages of the upflow deep-bed continuous backwash filters are:

- Sensitive to high solids-loading and process upsets (2-meter beds perform better than 1-meter beds).
- Airlift tubes can become clogged.
- Sand abrasion can wear through airlift tubes.
- A compressed air system and metering panels are required.
- Media can be lost during backwash unless careful attention is paid to air flow.
- Complex design compared to cloth-media filters.

Due to its smaller footprint and its good record of performance in Title 22 applications, the upflow deep-bed continuous backwash filter technology is recommended for further evaluation.

Recent experience at the Dublin San Ramon Services District (DSRSD) tertiary filtration project indicates that the very deep-bed (80-inch) upflow continuous backwash filter is preferable to the regular deep bed (40-inch) filter for secondary effluent from an activated sludge plant operating at a low SRT if full pretreatment (with clarification process) is not included. Pilot testing results showed that filter effluent quality using a 40-inch bed depth could not consistently meet Title 22 requirements when influent turbidity was high, even when coagulation and flocculation were provided. The 80-inch-deep filter provided consistent results, meeting Title 22 turbidity requirements following only coagulation and flocculation. The 40-inch-deep filter worked well with high-rate ballasted clarification (Actiflo®) as pretreatment. The 80-inch-deep filter was selected for implementation at DSRSD with coagulation and flocculation pretreatment.

Figure 4-12
Configuration of Upflow, Continuous Backwash Sand Filter
Parkson, DynaSand. Filter in steel vessel (left) and concrete tank (right)



4.5.2.3 Shallow-bed Traveling Bridge Filters

Shallow-bed traveling bridge filters use an 11-inch-deep bed of sand in a segmented bed arrangement, as shown on Figure 4-13. The backwash system is suspended from a bridge that moves across the filter bed.

Backwashing is accomplished by a pump that takes filtered water from the effluent channel and directs it back to the filter bed, forcing the water back up through the bed to dislodge trapped particles. A second pump, attached to a hood covering the segment being washed, draws waste backwash water into the hood and discharges it into a trough that returns the water to the head of the treatment plant or a sewer. Since the bridge travels continuously across the filter bed, this process produces a continuous and relatively small backwash stream. Because the backwash process is continuous, there is no need for backwash supply and waste equalization tanks.

The system requires relatively little pressure head, allowing it to fit within the hydraulic profiles of many treatment plants without additional intermediate pumping into the filters.

The advantages of shallow-bed traveling bridge filters are:

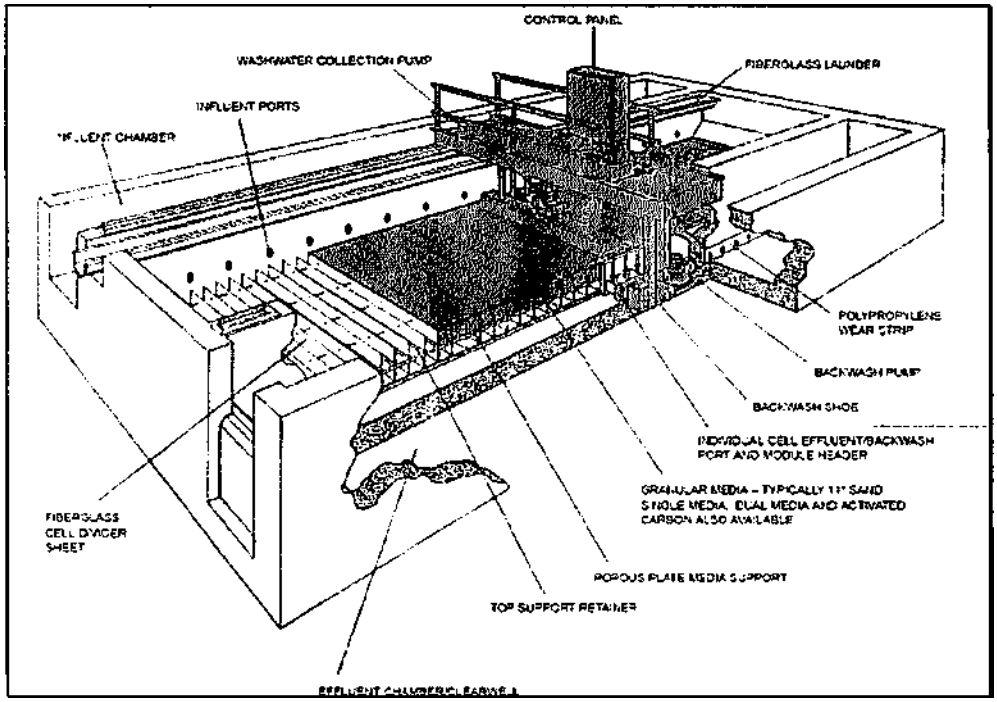
- Proven record of performance at municipal wastewater treatment plants in California and the U.S., but less than other technologies.
- Continuous operation (no shutdown during backwash).
- No backwash holding or equalization tanks required.
- Low power requirements.
- Low O&M requirements.

The disadvantages of shallow-bed traveling bridge filters are:

- Sensitive to high solids loading and process upsets.
- A compressed air system and metering panels are required.
- Relatively large footprint.
- Complex design compared to cloth-media filters.

Shallow-bed traveling bridge filters have similar advantages and disadvantages as the upflow continuous-backwash filters. However, because of its shallow media depth, DHS limits the hydraulic surface loading rate to 2 gpm/ft^2. This filtration technology is not desirable in this case because of the larger footprint required compared to the upflow filter technology. Moreover, it offers no cost or performance benefits compared to other technologies. Therefore, it is not recommended for further consideration.

Figure 4-13
Configuration of Traveling Bridge San Filter (*Aqua Aerobic System, Aqua ABF)*



4.5.2.4 Shallow Pulsed-bed Filter

The Hydro-Clear® pulsed-bed sand filtration system marketed by US Filter/Zimpro uses a unique underdrain system and a shallow bed (10-inches) of mono-media, fine-grained (0.45 mm) sand. The system includes the following subsystems:

- Pulse Mix® system
- Hydro-Scour® system
- Chem-Clean® system

The system is designed such that the filter media is "pulsed" periodically to remove accumulated solids. The Pulse Mix® system regenerates the media surface, which extends filter runs and keeps the system online during upsets. The Hydro-Scour® system is designed to backwash the media in 3.5 minutes using low power and water rates. The Chem-Clean® system allows semi-automatic media cleaning to remove grease and oil from the filter bed.

Although the net backwash water volume needed by shallow pulsed-bed filters is less than that of conventional filters, backwash supply and waste equalization tanks would still be needed for periodic backwash cycles.

The advantages of shallow-bed traveling bridge filters are:

- Proven record of performance at municipal wastewater treatment plants in California and the United States, but less than other technologies.
- Low power requirements.
- Low operation and maintenance requirements.

The disadvantages of shallow-bed traveling bridge filters are:

- Backwash supply and waste equalization tanks are required.
- Sensitive to high solids loading and process upsets.
- Shutdown for backwash required.
- A compressed air system and metering panels are required.
- Relatively large footprint.
- Complex design compared to cloth-media filters.

Shallow pulsed-bed filters have similar advantages and disadvantages as other sand filters. However, this filtration technology is not desirable in this case because of the larger footprint required compared to upflow filter technology. Moreover, it offers no cost or performance benefits compared to other technologies. This is a proprietary technology without an "equal" competitor. Because of the proprietary nature of this system and its large footprint, the system is not recommended for further consideration.

4.5.2.5 Cloth-media Filters

Cloth-media filters are relatively new to the wastewater industry in the United States but are gaining popularity because of their relatively low cost, compact footprint, and ease of operation. The technology originated in Europe and was brought to the United States in the early 1990s.

Cloth-media filters are relatively new in California, but they have been used extensively in other states for water reuse applications, especially in plants with high SRTs. However, pilot testing at DSRSD and other locations indicates that cloth-media filters are not reliable for filtering secondary effluent from activated sludge plants operating at low SRTs without full pretreatment.

The two cloth media filters approved for Title 22 applications are the AquaDisk by Aqua-Aerobic Systems and Hydrotech Discfilter by Veolia Kruger. These filters use a cloth fabric as the filtering medium, and they are typically arranged as vertical disks in concrete, fabricated steel or stainless-steel tanks, as shown on Figure 4-14. The flow path through the disks, disk submergence, cloth type, and backwash method are the principal differences between AquaDisk and Hydrotech Discfilter. However, from a design standpoint, these two filters can be considered equal since they have similar performance and can be accommodated by a single design.

Both systems use a series of vertical disks that support the filter cloth. The vertical orientation provides a large amount of filter area in a small footprint.

The AquaDisk filter medium consists of nylon fibers arranged in a pile construction on a polyester backing material. The nominal pore size of the media is 10 microns, and the media depth is approximately 3 to 5 mm. Each disk is made up of six pie-shaped sections that are mounted vertically to a common hollow tube that conveys filtered effluent from the tank. Under normal operation, the disks are completely submerged and stationary.

In the AquaDisk system, feed water flows through the filter media from the outside to the inside of the disks. The medium is washed by applying a vacuum to the outer surface of the disk surface. The vacuum draws water and retained solids off the media as the disks rotate. Solids that settle within the tank prior to reaching the filter media are removed by a sludge pump set to run on a timer.

The Hydrotech Discfilter medium consists of woven polyester fibers with a nominal pore size of 10 microns. The medium is essentially a microscreen supported by modular stainless-steel filter panels. The filter panels are mounted vertically to a common hollow tube that conveys feed water to each disk. Under normal operation, the disks are partially submerged and stationary. Wastewater flows from the inside to the outside of the disk. The filter is automatically backwashed by applying a high-pressure spray to the outside of the filter panels as the disk rotates, forcing retained solids and floatable materials into a backwash collection trough.

Both types of filter are backwashed automatically based on water level differential (or a timer) and maintain continuous filtration during backwash. Backwash water is supplied from filters that remain online, and typical backwash consumption is 1 to 3 percent of filter production. The media require relatively little cleaning energy, resulting in low backwash water volume without compromising filtration performance.

Although various cloth-media filtration systems are available in the market that could potentially be utilized to filter the combined effluent of the SFO WPCPs, the technology is relatively new with fewer installed facilities to consider its use at this time.

The advantages of cloth-media filtration are:

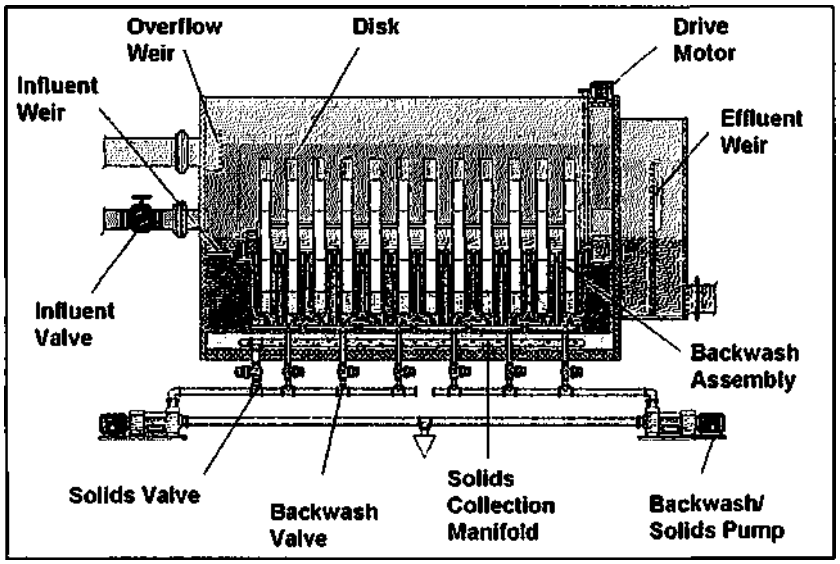
- Smallest footprint.
- Low capital cost.
- Continuous operation.
- Low backwash water consumption.
- No sand, underdrains, or airlift tubes to clog.
- High solids loading capacity.
- No backwash holding or equalization tanks required.
- Low power requirements.
- Low operation and maintenance requirements.
- Simple design.
- Simple to operate.

The disadvantages of the cloth-media filtration are:

- Less experience than other filter technologies, especially in California.
- Cloth media replacement recommended after about 6 years.
- Not reliable for low-SRT activated sludge effluent without pretreatment.

The AquaDisk worked well with Actiflo® as pretreatment in the DSRSD pilot tests. Hence, the cloth-media filtration may be a viable technology in this case. However, this technology will only be evaluated in combination with full pretreatment including clarification.

Figure 4-14
Configuration of Cloth-media Filter Aqua Aerobic Systems, AquaDisk



4.5.2.6 Fuzzy Filters

The Fuzzy Filter, marketed by Schreiber, is a high-rate filtration system that uses compressible, synthetic fiber spheres as the medium for filtration. Filter cells are square, and feed water flows upward through the medium. This technology has been approved to operate at hydraulic loading rates up to 30 gpm/ft^2.

The low density and high porosity of the medium results in more solids being captured per volume of media compared to conventional filters. Because of its compressibility, the porosity of the filter media bed can be altered to suit influent characteristics by mechanically compressing the filter bed. The filter medium is different from conventional filter media in that the water flows through the medium as opposed to flowing around the medium. These features permit relatively high hydraulic loading.

Air scouring is used during the wash cycle to clean the media. During the wash cycle, influent continues to enter the filter (filtered water is not required for washing) while an external blower supplies air in the bottom of the chamber to agitate the medium. The medium, which is retained between two perforated plates, is subjected to vigorous air scouring to free captured solids. Freed solids exit the filter continuously during the washing cycle. After the washing cycle, the medium is returned to its compressed state and filtration is resumed.

The Fuzzy Filter is a proprietary filtration technology without an "equal" competitor. The only known installation in California that produces recycled water under Title 22 is in Yountville. The Fuzzy Filter has similar advantages and disadvantages as cloth-media filters. However, because it is proprietary and lacks experience in California, it is not recommended for further consideration.

4.5.2.7 Non-membrane Filter Technologies Recommended for Further Evaluation

Based on the above discussion of non-membrane filtration technologies, two filtration technologies have been recommended for further evaluation. These are the upflow, continuous backwash sand filter and the cloth-media filter systems.

Upflow Continuous Backwash Filter Design Criteria. As discussed previously, there are two bed-depth options for upflow continuous-backwash filters. For alternatives with full pretreatment including clarification, 40-inch-deep beds would be used. Pretreatment may allow hydraulic loading up to 5 gpm/ft^2, the maximum allowed under Title 22.

For alternatives that include pretreatment consisting of coagulation and flocculation only, 80-inch-deep beds would be used. The surface loading rate would be designed more conservatively at 4 gpm/ ft^2 with one cell out of service (for redundancy).

Upflow continuous backwash filters are usually configured in 50-square-foot-modules, but larger modules are beginning to be used by some vendors to reduce costs.

Cloth Media Filter Design Criteria As discussed previously, cloth-media filters would only be used in conjunction with full pretreatment that includes clarification. With stable influent quality from full pretreatment, a maximum loading rate of 6 gpm/square feet can be used with one filter out of service (for redundancy). The filters usually have even number of disks, each having a filter area of 50 to 60 square feet.

4.5.3 Membrane Filtration Technologies

Membrane filtration is also approved under Title 22 for producing "disinfected tertiary recycled water," and it can be used to pretreat water prior to advanced treatment using RO as well.

4.5.3.1 Overview of Membrane Filtration Technologies

Low-pressure membrane filtration technologies, including microfiltration and ultrafiltration, are relatively new, especially in the wastewater industry. Nevertheless, use of membranes in water and wastewater treatment has grown rapidly during the past 5 to 6 years. This growth is due to improvements in membrane technology and system integrity, making low-pressure membranes more versatile and reliable than conventional filters for reducing turbidity and removing pathogens. Other reasons that membrane filtration has increased in popularity include its compact size, reductions in capital and operating costs, acceptance by regulatory agencies, and favorable experience at operating facilities. Recent major uses of MF and ultrafiltration include treatment of potable water, water recycling, pretreatment for RO, and liquids/solids separation in MBRs.

Low-pressure membrane systems can include spiral-wound, hollow-fiber, sheet, and tubular configurations. However, for drinking water and water recycling applications, hollow-fiber membranes predominate. The main advantages of hollow-fiber MF/UF include the high packing density and capability to tolerate frequent cleaning cycles. Both immersed (Figure 4-15) and pressurized (Figure 4-16) hollow-fiber MF/UF systems are approved for use under Title 22. For MBR applications, hollow-fiber and plate technologies can be used.

MF and UF are designed to remove particulate matter by size exclusion. Particles ranging from 0.05 to 1.0 um can be removed by MF, while particles ranging from 0.003 to 0.1 um can be removed by UF. Unlike high-pressure membrane processes such as nanofiltration (NF) and RO, MF and UF usually operate in the range of 3 to 50 pounds per square inch gauge (psig).

When membrane systems are used for producing recycled water, Title 22 requires that the filtered water turbidity not exceed 0.2 NTU more than 5 percent of the time within a 24-hour period and 0.5 NTU at any time. In addition, membrane integrity testing is required.

Figure 4-15
Configuration of Immersed MF/UF Membrane Filtration *Zenon, Zeeweed*

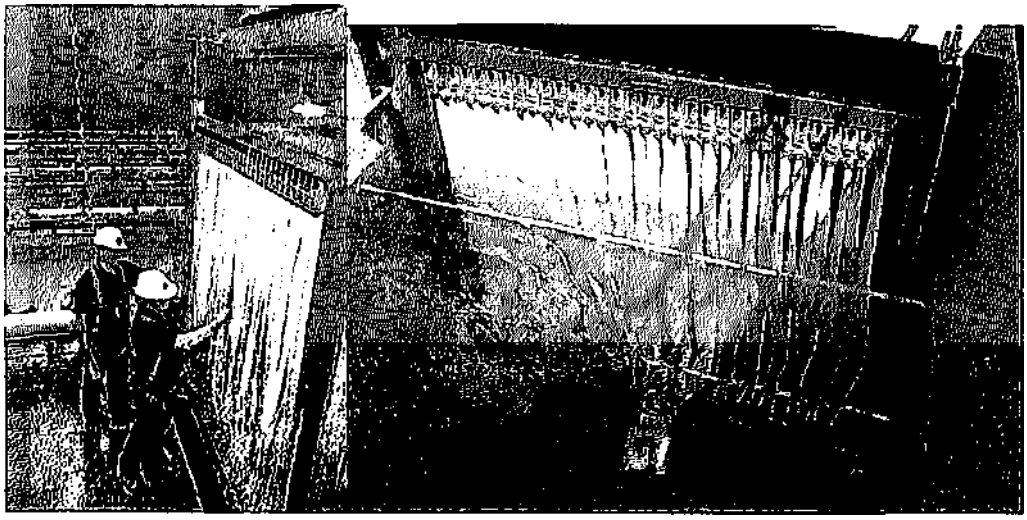
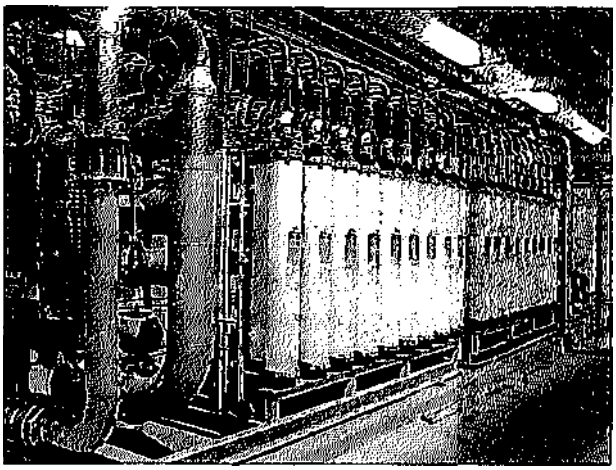


Figure 4-16
Configuration of Pressurized MF/UF Membrane



4.5.3.2 Membrane Filtration Technologies Accepted under Title 22

MF/UF technologies from four vendors, including Zenon (General Electric), US Filter, Pall, and Ionics (General Electric), are accepted under Title 22. Each vendor provides unique membrane and backwash systems. Two of the vendors (Zenon and US Filter) supply immersed membrane systems that use vacuum force to pull clean water through hollow-fiber membranes. Figure 4-17 shows a general process flow diagram of an immersed membrane system. Others (including US Filter) supply membrane cartridges incorporating thousands of hollow-fibers or capillary tubes installed inside pressurized vessels. Figure 4-18 shows a general process flow diagram of a pressurized low-pressure membrane system.

In hollow-fiber systems, feed water can flow from the outside to the inside or from the inside to the outside.

The four available systems are described below, and Table 4-3 provides a summary of the systems.

Table 4-3
Membrane Filtration Technologies Approved under Title 22

Vendor/Technology	MFor UF	Pressurized or Immersed	Membrane Material	Pore Size or MWCO	Backwash Procedures	Flow Pattern
US Filter/Memcor/CMF and CMF-S	MF	Both	PPL/PVDF	0.2 pm	Air/water	Outside-ln
Pall (Asashi)/ Mlcroza™	MF/UF	Pressurized	PVDF/PAN	13,000 or 80,000	Air/water	Outside-ln
GE Ionics (NorityXIGA™	UF	Pressurized	PVP/PES	0.03 pm	Water only	Inside-Out

Figure 4-17
Schematic Process Flow Diagram of Immersed MF/UF Membrane Filtration

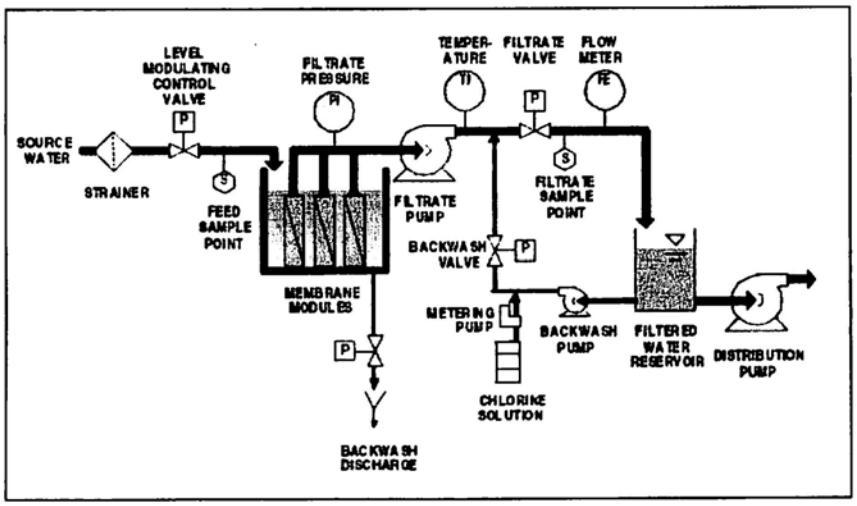
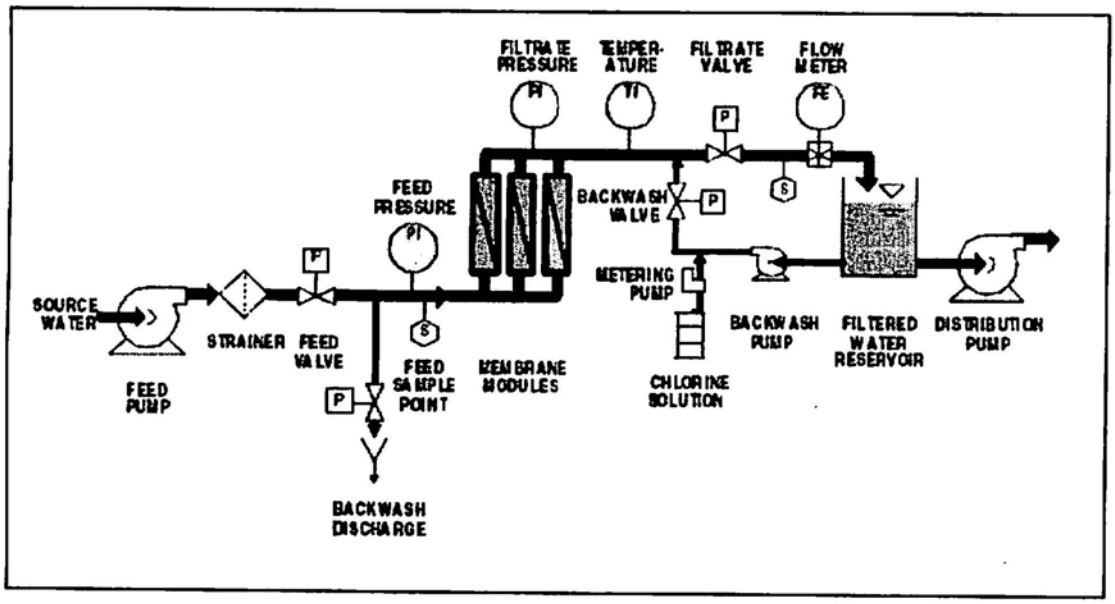


Figure 4-18
Schematic Process Flow Diagram of Pressurized MF/UF Membrane Filtration



Zenon ZeeWeed® (Immersed). ZeeWeed® is a patented immersed membrane technology manufactured by Zenon Environmental, which was recently acquired by General Electric. Like other immersed membrane technologies, the ZeeWeed® process operates under a partial vacuum (-3 to -12 psig). ZeeWeed® membranes have a nominal pore size of 0.036 microns, which qualifies as UF.

The ZeeWeed® 500 system, which is typically used in wastewater applications, uses a reinforced fiber backing with a membrane cast to the surface. The outer and inner diameters of the hollow fibers are 1.9 and 0.9 mm, respectively.

Because ZeeWeed® operates at relatively low pressure; energy consumption is low compared to pressurized systems. In addition, the rate of fouling is typically lower compared to pressurized systems because the pressure drop across the membranes {i.e., transmembrane pressure (TMP)}is lower. Since flow passes from outside to inside the hollow fibers, the risk of plugging flow channels inside the membrane tubes is relatively low.

The membrane material, polyvinylidene difluoride (PVDF), can tolerate chlorine at concentrations of up to 2,000 mg/L, which facilitates periodic cleaning to remove biofilm, restore flux, and reduce TMP.

Immersed membrane systems can be designed within relatively small spaces, and they can be installed in existing tanks (when available) to reduce costs. Individual membrane strands are packaged in modules. Each module is integrated into a cassette of either eight or 22 ZeeWeed® membrane modules. The cassettes are connected to a main permeate header pipe in trains of up to 15 cassettes.

During operation, the membrane fibers are scoured continuously or periodically by coarse aeration to remove solids that accumulate on membrane fibers. Aeration intensity and frequency can be adjusted to suit site-specific conditions and to save energy. By removing solids, aeration allows the system to operate at higher flux rates and lower TMP. In addition, air scouring minimizes concentration polarization, which is recognized as a significant mechanism of membrane fouling that can contribute to poor permeate quality. Aeration allows the Zee Weed® system to handle very high TSS concentrations. In MBR systems, the TSS concentration can be as high as 15,000 mg/L.

The ZeeWeed® membrane system also includes an automatic backpulse system that is used to remove solids from the surfaces of the membrane fibers. During a backpulse cycle, permeate is pumped backwards through the membrane fibers for a few seconds. The frequency of backpulse cycles can be adjusted to suit site-specific conditions.

A clean-in-place (CIP) system is used as a supplement to the backpulse system. The CIP system provides more thorough cleaning when needed to recover flux or reduce TMP. The CIP system can apply sodium hypochlorite solution to remove biological fouling or acid solution to remove mineral fouling.

For water with relatively low turbidity (i.e., secondary effluent or raw water with full pretreatment), Zenon sometimes recommends its newer and less expensive ZeeWeed® 1000 system. The ZeeWeed® 1000 system has similar filtration characteristics as the ZeeWeed® 500 system, but it is oriented horizontally instead of vertically. In addition, membrane fibers in the ZeeWeed® 1000 system do not have fiber backing. The ZeeWeed® 1000 membrane system does not need air scouring except during backpulse cycles, so the system uses less energy. Both ZeeWeed 500 and ZeeWeed 1000 are accepted for Title 22 applications.

US Filter (now Siemens**) Memcor® (Immersed and Pressurized).** US Filter's Memcor pressurized (encased) Continuous microfiltration (CMF) system was the most popular low-pressure membrane process when membrane filtration was first introduced to the water and wastewater industries. It was primarily used as pretreatment for RO in municipal wastewater reclamation projects and for potable water treatment.

The CMF system uses hollow-fiber membranes that operate in the 'outside-in' mode. A patented compressed-air backwash system is used to control fouling, so the CMF system can handle relatively high TSS concentrations.

The membrane fibers are usually made of polypropylene with a pore size of 0.2 microns. Polypropylene cannot tolerate free chlorine, so its use is limited in applications where free chlorine is used to control biofouling. However, in applications where the feedwater has a high concentration of ammonia, chlorine and ammonia can be combined to form chloramines that do not harm polypropylene membranes but are still effective for controlling biofouling.

US Filter later developed the Continuous Microfiltration submerged (CMF-S) system that is similar to the immersed system offered by Zenon. In addition, US Filter now offers membranes constructed of PVDF in both the CMF and CMF-S systems to overcome intolerance to free chlorine.

In the CMF-S system, the hollow-fiber membrane modules are immersed into the water or wastewater to be treated. A vacuum pump draws permeate to the inside of the fibers, leaving the

solids in the tank. Aeration is used to scour the membranes during backwash cycles. The CMF-S system is preferred to the CMF system in larger projects (>10 mgd) because of favorable economics and reduced space requirements.

Pall (Asahl) (Pressurized). Pall Corporation offers pressurized MF and UF systems that use Microza™ hollow-fiber membranes supplied by Asahi Chemical Industries. Pall MF/UF systems have been widely used in drinking water treatment, municipal wastewater reclamation, and as pretreatment ahead of RO during the past several years. Microza™ MF and UF systems are used as pretreatment for RO/NF processes in the City of Chandler, Arizona to treat semiconductor wastewater. A Pall MF system was installed near the Santa Rosa Airport to treat lagoon wastewater to produce Title 22-quality water.

The hollow fibers of the MF system have inner and outer diameters of 0.7 mm and 1.3 mm, respectively. The pore size rating is 0.1 microns. The material of the MF fibers is PVDF, and the material of the pressurized housing is PVC.

The UF hollow fibers have inner and outer diameters of 0.8 mm and 1.4 mm, respectively. The pore size rating is 13,000 to 80,000 Daltons in molecular weight cutoff. The material of the UF fibers is polyacrylonitrile and the material of the pressurized housing is PVC. PVDF is more chlorine resistant than polyacrylonitrile, but polyacrylonitrile can withstand free chlorine at concentrations up to 1,000 mg/L.

The Microza™ MF/UF modules operate in an 'outside-in' mode with a small amount of recirculation past the outside of the hollow fibers. The recirculation helps prevent accumulation of deposits on membrane surfaces and evenly distribute flow through the module.

Periodically, each rack goes through a backwash cycle consisting of simultaneous air scrub - reverse filtration. Compressed air and backwash water loosen and dislodge particles collected on the membrane fibers. If biological fouling is to be removed, a small amount of chlorine is injected into the reverse filtration water as it is being fed into the modules.

The modules may be cleaned periodically through a process called enhanced flux maintenance that uses 1 percent sodium hydroxide (NaOH) and 0.1 percent sodium hypochlorite (NaOCl) in the backwash water. Enhanced flux maintenance can be initiated daily or weekly to reduce TMP rise, reduce feed-pressure requirements, and prolong periods between full chemical cleaning cycles.

When needed, a more complete cleaning cycle can be performed using a CIP system. Chemicals used in the CIP system can include NaOCl, NaOH, and mild acid, depending on site-specific requirements. The maximum allowable concentrations for NaOCl are 5,000 mg/L for PVDF and 1,000 mg/L for ployacrylomitrile membranes. Maximum caustic strengths are IN for PVDF and 0.1N for ployacrylomitrile membranes; both types can withstand IN strength acid.

GE Ionics/Norlt UF (Pressurized). GE Ionics supplies UF systems using Norit XIGA™ UF membrane systems. The XIGA™ element consists of a pressurized PVC cartridge with a central permeate tube that contains membrane capillaries made from a blend of polyethersulphone and polyvinylpyrrolidone.

Unlike other MF and UF systems approved for Title 22 applications, this system uses 'inside-out' crossflow membrane elements that are analogous to RO membrane elements. Up to four 1.5-meter

elements are installed inside each pressure vessel. The membranes fibers are typically 0.8 or 1.5 mm in diameter, and each element has a surface area of 22 or 35 square meters.

Like other MF/UF systems, this system requires periodic cleaning to reduce TMP. Clean permeate water is forced from the outside of the capillaries to the inside to transport accumulated solids out of the capillaries. The maximum allowable TMP during backwash is 44 psi, and the maximum allowable flux rate is about 150 gfd. The duration of a backwash cycle is generally 30 to 40 seconds, and the frequency is typically about 15 to 20 minutes.

Periodic cleaning using disinfectants like peroxide or hypochlorite is recommended to control bacterial growth. In addition, cleaning using chemicals like sodium hydroxide, ethylenediaminetetraacetic acid, citric acid, etc. may be used when needed.

If backwashing alone is insufficient to reduce TMP, chemically-enhanced backwash can be used. The chemically-enhanced backwash mode consists of a normal backwash procedure, a backwash cycle with chemicals, a soaking period, and another backwash cycle to flush-out the chemicals.

The XIGA™ UF technology is used as pretreatment for RO for a large municipal wastewater treatment and reuse project near Kuwait City. Tine treatment plant in Sulaibiya, Kuwait, which was commissioned in 2005, is the largest membrane-based municipal effluent reuse plant in the world (100 mgd or 380 ML/d).

4.5.3.3 Summary of Membrane Filtration Technologies

Each of the membrane filtration processes approved under Title 22 are suitable for producing recycled water for irrigating landscaping and/or for pretreatment ahead of RO. However, immersed systems have become more common in recent years, especially in larger applications. Immersed systems are advantageous because they use less energy and are less prone to fouling. Therefore, if MF or UF is to be used, the immersed systems provided by Zenon and US Filter are preferable.

4.6 Disinfection Technologies

Recycled water that is used for irrigating landscaping, recharging unrestricted recreational impounds without conducting special monitoring, or commercial purposes must be disinfected according to the following standards set forth in Title 22:

> *"Disinfected tertiary recycled water" means a filtered and subsequently disinfected wastewater that meets the following criteria:*
>
> *(a) The filtered wastewater has been disinfected by either:*
>
> *(1) A chlorine disinfection process following filtration that provides a CT (the product of total chlorine residual and modal contact time measured at the same point) value of not less than 450 milligram-minutes per liter at all times with a modal contact time of at least 90 minutes, based on peak dry weather design flow; or*
>
> *(2) A disinfection process that, when combined with the filtration process, has been demonstrated to inactivate and/or remove 99.999 percent of the plaqueforming units of F-specific bacteriophage MS2, or polio virus in the wastewater. A virus that is at least as resistant to disinfection as polio virus may be used for purposes of the demonstration.*
>
> *(b) The median concentration of total colifonn bacteria measured in the disinfected effluent does not exceed an MPN of 2.2 per 100 milliliters utilizing the bacteriological results of the last seven days for which analyses have been completed and the number of total coliform bacteria does not exceed an MPN of 23 per 100 milliliters in more than one sample in any 30 day period. No sample shall exceed an MPN of 240 total Coliform bacteria per 100 milliliters.*

These requirements can be satisfied using disinfection processes that use chlorine or irradiation by UV light. Other disinfection processes may be feasible as well, but only chlorine-based and UV systems are currently in use for Title 22 applications.

Chlorine disinfection processes can use gaseous chlorine or chlorine in another form such as sodium hypochlorite or calcium hypochlorite. Gaseous chlorine is being phased out by the Department of Homeland Security and most water and wastewater utilities for security reasons and because it is relatively dangerous to handle. Therefore, gaseous chlorine is excluded from the alternatives evaluated below. In addition, calcium hypochlorite systems are generally not practical at large facilities. Therefore, only sodium hypochlorite and UV irradiation are considered viable options for producing "tertiary disinfected recycled water."

Recycled water used as an indirect potable water supply must undergo additional treatment that is described below.

4.6.1 Disinfection Using Sodium Hypochlorite

Options for disinfection using sodium hypochlorite solution include:

- Delivery of high-strength sodium hypochlorite solution in bulk.
- Onsite generation of low-strength sodium hypochlorite solution.
- Onsite generation of high-strength sodium hypochlorite solution.

Each of these options would include chlorine contact tanks where sodium hypochlorite solution would be mixed with filtered water and held for a specified period.

4.6.1.1 Delivery of High-strength Sodium Hypochlorite in Bulk

Sodium hypochlorite solution can be delivered to the treatment facility in bulk at a minimum concentration of 12.5 percent. The solution would be stored in tanks and delivered to the injection point using metering pumps. The system would be constructed inside a containment basin as required by fire codes. Similar systems are in operation at numerous water and wastewater treatment facilities across the United States.

4.6.1.2 Onsite Generation of Low-strength Sodium Hypochlorite

In an onsite generation system, an electric current is applied to a concentrated brine solution to generate low-strength (0.8 percent) sodium hypochlorite solution. The system uses high-purity, food-grade sodium chloride salt dissolved in softened water. The resulting brine mixture is fed to hypochlorite generators. Approximately 3.5 pounds of salt are required to produce 1 pound of available chlorine. The waste product, hydrogen gas, is normally vented to the atmosphere. The entire flow stream, consisting of remaining brine solution and hypochlorite, is injected into the filtered water stream.

A hypochlorite generating system would consist of a water softener, salt storage/brine tanks, brine feed pumps, hypochlorite generators, hypochlorite day tanks, metering pumps, and an acid cleaning system for the generators' electrolytic cells. The generation system would need to be connected to a standby power generator to ensure continuous power supply.

The electrolytic cells must be cleaned regularly with an acid solution. The cleaning process produces a concentrated waste acid byproduct. Typically, the cells require cleaning after 2,000 to 4,000 hours of service. The cleaning frequency is dependent on the salt purity and water quality. In addition, the electrolytic cells require replacement about every 3 to 5 years, depending on the hours of operation. To minimize electrode fouling, soft water (maximum hardness of 25 mg/L as CaCCb) that is free of trace organics and metals is recommended to create the brine feed.

4.6.1.3 Onsite Generation of High-strength Sodium Hypochlorite Solution

A company called Electrolytic Technologies Corporation has developed a new technology to produce either high-strength (-12 percent) sodium hypochlorite solution or separate streams of chlorine gas and sodium hydroxide on site. Advantages over older technology that produces low-strength sodium hypochlorite solution (~1 percent) may include reduction in chloride and sodium added to treated water and lower maintenance requirements.

The system, which is called Klorigen, combines electrolysis of brine with membrane technology to deliver chlorine gas at atmospheric pressure and caustic soda at a 15 percent concentration.

The process may combine these two products to produce sodium hypochlorite at a 10 to 12 percent concentration. Systems are available to produce 55 to 1,200 gallons per day of 10 to 12 percent sodium hypochlorite solution. Since the process entails generation of chlorine gas, it is not considered a viable option.

4.6.2 Disinfection Using UV Irradiation

UV disinfection involves the use of ultraviolet light generated by lamps immersed in water to deactivate pathogens. The principal mechanism for UV disinfection is different from the oxidant-based disinfectants. UV disinfection uses photochemical energy to prevent cellular proteins and nucleic acids (i.e., DNA and RNA) from further replication. The germicidal effect of UV light occurs through the dimerization of pyrimidine nucleobases (e.g., thymine) on the DNA molecules to distort the normal helical structure and prevent cell replication. A cell that cannot replicate also cannot infect. The UV electromagnetic waves range from 40 to 400 nm long (between the X-ray and visible light spectrums). The germicidal UV light wavelengths range from 200 to 300 nm, with the optimum germicidal effect occurring at 253.7 nm.

UV electromagnetic energy typically is generated by the flow of electrons from an electrical source through ionized mercury vapor in a lamp. Several manufacturers have developed systems that align UV lamps in vessels or channels to provide UV light in the germicidal range for inactivation of bacteria, viruses, and other microorganisms. The UV lamps are similar to household fluorescent lamps except that fluorescent lamps are coated with phosphorous that converts the UV light to visible light. Ballasts (i.e., transformers) that control the power to the UV lamps are either electronic or electromagnetic. Electronic ballasts offer several potential advantages including lower lamp operating temperatures, higher efficiency, and longer life.

4.6.2.1 Parameters that Affect Design and Performance of UV Disinfection Systems

Research indicates that typical water quality parameters such as pH, temperature, alkalinity, and total inorganic carbon do not appear to affect the overall effectiveness of UV disinfection. Hardness affects the rate of lamp fouling, but the automatic lamp cleaning systems incorporated in newer UV equipment have minimized the impact of hardness on system design and operation.

The effectiveness of UV disinfection is mainly impacted by water quality parameters that prevent UV energy from reaching target micro-organisms. Particles, turbidity, and suspended solids may shield microorganisms from UV light or scatter UV light to prevent it from reaching target microorganisms. Some organic compounds (e.g., phenols) and inorganics (e.g., iron) absorb UV energy and reduce the UV transmittance of the water being treated. Thus, turbidity and UV transmittance are commonly used as process controls at UV facilities.

UV transmittance is measured by a UV-range spectrophotometer set at a wavelength of 253.7 nm using a 1-cm-thick layer of water. UV transmittance is related to UV absorbance (A) at the same wavelength by the equation:

$$Percent\ Transmittance = 100 \times 10^{A}$$

For example, a UV absorbance of 0.022 per cm corresponds to a water percent transmittance of 95 percent (i.e., at 1 cm from the UV lamp, only 95 percent of lamp output is left). Similarly, 90 percent UV transmittance is equivalent to UV absorbance of 0.046 per cm.

The footprint size and the associated cost of a UV disinfection system depend on the selected UV design dose. Because UV transmittance is a direct measure of the capacity of the water to transmit UV light, the required size and cost of a U V system also depend directly on the design value of UV transmittance.

4.6.2.2 Regulatory Issues Related to UV Disinfection

Guidelines for designing UV disinfection facilities have been developed to provide direction to regulatory staff of the Water Board and the DHS, the agencies responsible for approving UV disinfection for water recycling applications. An independent panel of experts, convened by the National Water Research Institute (NWRI) and the DHS, developed the guidelines for UV disinfection. The *Treatment Technology Report for Recycled Water* issued by the DHS (March 2006) contains the following excerpt regarding use of UV disinfection to treat recycled water:

> *UV Disinfection Guidelines were first published in 1993 by the National Water Research Institute. Since that time, the field of ultraviolet disinfection has taken great strides forward. As a result of the progress made in understanding the UV disinfection process, the CDHS and the NWRI agreed that it was time to revise and update the guidelines. NWRI and the American Water Works Association Research Foundation [AWWARF) pooled their resources in order to revise the original guidelines, which noiv cover water recycling and drinking water UV disinfection applications. As a result of these efforts the "Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse" were published by NWRI/AWWARF in December 2000 and revised as a Second Edition dated May 2003. CDHS endorses the May 2003 Guidelines and refers to them when evaluating UV disinfection proposals. One major recommendation of the guidelines is that all UV equipment (including previously approved equipment) be tested and validated under these new guidelines before being accepted by the Department. For existing systems approved under earlier guidelines, documentation of compliance with the May 2003 guidelines should be provided when permits issued by the Regional Water Quality Control Boards come up for renewal. It is believed that existing UV disinfection systems that were properly designed should comply with the elements of the revised guidelines.*

According to the technology report, the following systems have been validated by DHS under the 2003 NWRI/AWWARF guidelines:

- Trojan Technologies, Inc., UV 3000 Plus (low pressure/high intensity)
- Wedeco/Ideal Horizons TAK 55 HP (low pressure/high intensity)
- Infilco Degremont, Aquaray 40 Vertical Lamp System (low pressure/low intensity)
- Infilco Degremont, Aquaray 40 High Output Vertical Lamp System (low pressure/high intensity)

Each of these systems may be suitable for disinfecting recycled water produced using the filtration alternatives described previously. However, demonstration testing may be required to confirm that UV disinfection can be used following granular-media or cloth-media filtration, which will produce filtrate with relatively low transmittance. Use of UV disinfection following membrane filtration is expected to be feasible.

4.6.2.3 Configuration of UV Disinfection Systems

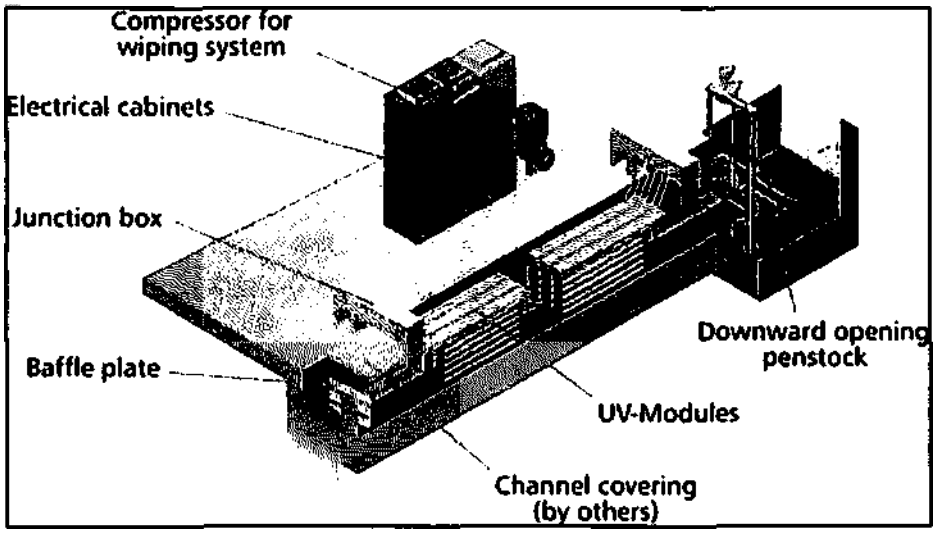
Mercury vapor lamps are used in all of the validated systems to produce UV light. Low-pressure systems operate at either 10^3 to 10^2 torr. Medium-pressure systems, which operate at 10^2 to 10^3 torr, have not been validated under the 2003 NWRI/AWWARF guidelines. These two ranges of operating pressure provide the highest conversion of electrical energy to radiation. Low-pressure mercury lamps are more efficient in producing germicidal UV radiation, but the total UV radiation is much weaker compared to medium-pressure lamps. The low-pressure, high-intensity mercury lamps have special design features to maintain mercury pressure at an optimum level under high discharge currents.

UV disinfection systems can be configured in three reactor designs: open-channel, closed-chamber, and Teflon-tube. However, all of the technologies validated to date under the 2003 NWRI/AWWARF guidelines use an open-channel configuration as shown in Figure 4-20. Orientation of the lamps varies within the open channels, with the lamps oriented either horizontally and parallel to flow or vertically and perpendicular to flow.

In a horizontal UV system, the lamps are grouped in modules with lamps spaced about 3 inches apart (center to center). The number of lamps needed in each module depends on the depth of the channel and varies from 2 to 16 lamps. Two to 24 modules can be used to span the width of a channel. The set of modules spanning the channel width comprises a lamp bank. Lamps are typically 64 inches long.
In a vertical UV system, each vertical lamp module contains five staggered rows, with eight lamps in each row. Within each row, the lamps are spaced 2.8 inches apart (center to center). The rows are spaced 5 inches apart (center to center) in the direction of flow. The number of modules in the lamp bank spanning the channel varies from one to four. An air-scouring system is provided with the Infilco Degremont system to reduce fouling. Typically, full-scale systems employ 64-inch-long lamps.

Cleaning of the lamp sleeves can be accomplished by taking UV banks out of service and moving them to a cleaning basin using an overhead crane. Once inside the cleaning basin, the lamps are agitated with a cleaning solution. A wash-down area is provided adjacent to the cleaning basin for wiping or rinsing purposes.

Figure 4-20
Configuration of Open-channel UV Disinfection Process



4.6.2.4 Comparison of UV Disinfection Alternatives

A vertical UV system has the advantage of being more compact than a horizontal UV system because vertical systems use a deeper channel. In addition, vertical systems pace flow well and lamps are easier to replace. Relatively high headloss is the main concern with vertical UV systems because higher headloss results in a larger variation of water level under the full range of flow. Conversely, increased headloss results in more mixing and possibly better disinfection.

The low-pressure, low-intensity UV system provided by Infilco Degremont requires manual cleaning of the lamps in a separate tank filled with an acid solution. This manual cleaning system requires a crane to move the modules between the UV channels and the cleaning tank. Conversely, the three low-pressure, high-intensity UV systems have automatic lamp cleaning mechanisms that minimize the need for manual cleaning.

Another disadvantage of the Infilco Degremont low-pressure, low-intensity UV system is that it requires three times more lamps than the low-pressure, high-intensity UV systems. Moreover, low-pressure, high-intensity UV systems are more energy efficient.

Given these relative advantages and disadvantages, low-pressure/high-intensity UV disinfection appears to be the best UV disinfection option. However, if UV disinfection is used, a relatively small sodium hypochlorite injection system would still be required to maintain chlorine residual in the recycled water distribution system to prevent regrowth of pathogens.

4.6.3 Summary of Disinfection Alternatives

Each of the sodium hypochlorite and UV disinfection alternatives described above can be used effectively. The major advantage of UV disinfection compared to sodium hypochlorite is that a UV system can be much smaller because a large contact tank is not required. On the other hand, UV disinfection uses significantly more energy compared to sodium hypochlorite processes, particularly when UV disinfection is used to treat water from granular-media and cloth-media filters.

Based on the initial screening evaluation, the following alternatives were retained for further evaluation:

- Imported bulk sodium hypochlorite solution.
- On-site generation of low-strength sodium hypochlorite solution.
- Low-pressure/high-intensity UV irradiation.

4.7 Membrane Bioreactor Technology

MBR technology could be used to produce recycled water from raw sewage at a new satellite treatment facility. The new facility would consist of the following unit processes:

- Preliminary treatment using fine screens and grit removal tanks.
- Biological and chemical treatment in aeration tanks.
- Immersed microfiltration or ultrafiltration in membrane tanks.
- Disinfection using chlorine contact or UV irradiation.
- Odor control.

An MBR system would incorporate one of the immersed MF/UF systems described above. In addition to providing membrane filtration, MBR systems can provide biological and chemical treatment using a combination of a hybrid suspended-growth activated sludge process with membrane filtration to separate solids and water.

The benefits of MBR systems include excellent effluent quality (TSS <1 mg/L and turbidity <0.2 NTU) and a small footprint compared to a conventional treatment process.

There are six manufacturers of membrane equipment for use in MBR systems, but only three are currently approved for Title 22 applications. Zenon has been the predominant provider in North America so far, but US Filter and Kubota also provide MBR systems that are approved under Title 22. The Kubota system is marketed by Enviroquip in the United States. Most offer a choice of purchasing the equipment only or of purchasing a package that includes equipment plus process design and performance responsibility.

Most MBR systems constructed to date have been packaged systems with capacities below 1 mgd. In larger plants, process design and equipment supply responsibilities are normally kept separate.

MBR systems rely on numerous ancillary processes including screening, grit removal, flow equalization, biological and chemical treatment, aeration for mixing, biological growth, and scouring membrane fibers, disinfection, and standby power.

4.7.1 Screening

Membrane filtration equipment requires fine screening to avoid fouling and damage by particles with a diameter greater than a 2-mm. Rotary drum or perforated traveling band screens are normally used. Screenings would be washed, dewatered, and conveyed to a roll-off waste container. The facility could be enclosed to minimize odors and prolong equipment life.

4.7.2 Grit Removal

Grit removal is necessary to protect the membrane fibers and prevent grit from collecting in the aeration tanks. A vortex or aerated grit chamber would be necessary to remove the grit. Grit could be washed and conveyed to a roll-off waste container.

4.7.3 Oxidation and Membrane Filtration

MBRs combine biological treatment and membrane filtration. Biological treatment occurs in aeration tanks where flow from the preliminary treatment processes is mixed and aerated with mixed-liquor that is recycled from the membrane tanks. Flow from the aeration tanks passes into the membrane tanks where some of the water is separated from the mixed liquor through membrane filtration. The remainder of the mixed liquor is recycled back to the aeration tanks. Alum can be added to the mixed-liquor to remove phosphorous through chemical precipitation.

The MBR process is analogous to conventional activated sludge processes. The membrane tanks are like secondary clarifiers in that they are used to separate solids from treated water. However, unlike a conventional activated sludge process, MBRs operate a relatively high mixed liquor suspended solids concentrations (8,000 to 12,000 mg/L). MBRs also operate at much higher mixed liquor recycle rates (typically around 400 percent of influent flow). Because MBRs use membrane filtration, the effluent quality is generally much better than effluent from conventional secondary clarifiers.

As with conventional activated sludge processes, aeration blowers and diffusers are used to provide oxygen for aerobic biological growth and for mixing/agitating the mixed liquor. Likewise, recyle pumps are used to return biomass to the aeration tanks, and waste pumps are used to remove excess solids.

The membrane systems consist of immersed membrane UF/MF like those previously described. The membranes act as molecular filters removing solids, bacteria, and some viruses. The membranes are immersed in the mixed liquor and filtration is achieved by drawing water through the membranes under a vacuum produced by permeate pumps.

Aeration plays a critical role in the operation of some types of membrane equipment. For most equipment, coarse bubble airflow is introduced to the bottom of the membrane modules producing turbulence that scours the external surface of the hollow fibers, thus transferring rejected solids away from the membrane surface. Air scouring reduces the need for periodic cleaning of the membranes.

4.7.4 Disinfection

Disinfection of the filtrate would be required in accordance with Title 22 guidelines. Any of the disinfection technologies described previously could be used.

4.7.5 Summary of MBR Technology

MBR technology is feasible for producing recycled water from raw sewage at a satellite treatment facility. MBR technology is also suitable for pretreatment of water to be processed through an RO system. However, the cost of satellite MBR treatment will be much higher compared to processes designed to treat secondary effluent from the SFO WPCP. Because of the high cost of MBR treatment, it is not recommended.

4.8 Advanced Treatment

Recycled water for use by UAL co-generation facility and Airport central cooling towers may require advanced treatment to remove nutrients and other constituents of concern. The advanced treatment will reduce the TSS and TDS and lower the conductivity of the Tertiary treated water to an acceptable level to prevent calcium carbonate scaling and corrosion.

4.8.1 Advanced Treatment Technology

Appendix C provides a summary of advanced treatment of recycled water by various agencies. Advanced treatment facilities approved to date by DHS include:

- Pretreatment systems.
- High-pressure pumps and RO membrane assemblies.
- CIP system.
- Post-treatment system.
- Advanced oxidation.

4.8.1.1 Pretreatment Systems

Pretreatment is necessary to reduce potential for fouling of RO membranes by mineral precipitation and microbial growth. Sulfuric acid and scale inhibitors are normally injected into the feed water for this purpose.

4.8.1.2 Sulfuric Acid

The pH of the feed water must be controlled to prevent precipitation of minerals such as calcium carbonate. Sulfuric acid can be added to the feed water to react with alkalinity, lower the pH, and reduce precipitation. The required acid dosage may be in the range of 35 to 45 mg/L, but testing is required to determine the optimal the dose.

4.8.1.3 Scale Inhibitors

Scale inhibitors are polymeric substances that interfere with mineral precipitation either by inhibiting the formation of mineral crystals or by limiting the extent of crystal growth. Modern inhibitors are effective for a range of sparingly soluble minerals, including carbonates, sulfates, and silica. Limiting scaling/fouling parameters may include barium sulfate, calcium sulfate, calcium fluoride, strontium sulfate, and silica, but additional water quality data are needed before a scale inhibitor can be selected.

4.8.1.4 High-pressure Pumps and Membrane Assembly

RO is a pressure-driven membrane-separation process that removes dissolved inorganic and organic compounds. RO systems for wastewater applications typically have three stages to recover of 80 to 85 percent of the feed water. Concentrate from the first stage is applied to a second stage, and the concentrate from the second stage applied be applied to a third stage. The flux rate is expected to be about 10 gfd at a water temperature of 21 "C, but additional testing is required to establish these parameters.

Pretreated water is fed continuously to the RO system at elevated pressure (150 to 200 psig) by a set of high-pressure feed pumps. Energy savings may be realized by adding pumping between RO stages. The required feed pressure varies depending on the dissolved solids content of the

feed water (i.e., osmotic potential), membrane properties, and the water temperature. RO feed pumps are usually equipped with variable speed drives to maintain constant flux.

Evaluation of RO Technology Alternatives - RO technology is becoming more popular as a treatment process to produce high-quality water. Historically, RO systems have used standardized 8-inch-diameter by 40-inch-long elements (8-inch by 40-inch). However, conventional 8-inch by 40-inch elements may be more expensive than newer 16-inch-diameter elements, depending on the size of the facility.

In 2004, a consortium of RO membrane element manufacturers prepared a study that concluded that the 16-inch-diameter by 40-inch-long elements could be more cost effective. The consortium was composed of the following membrane element manufacturers:

- Filmtec Corporation
- Hydranautics
- Toray Membrane America, Inc.
- Trisep Corporation

Although the consortium found that 16-inch elements are preferable, a schedule for producing 16-inch elements was not established. The latest information indicates that the 16-inch elements from the original consortium are still in pilot testing stage and may not be available before late 2009. In addition to the original consortium, DOW Water solution, a unit of DOW Chemical and Koch Membrane Systems, Inc has piloted a 16” and 18” (respectively) diameter membrane and are ready for production by late 2009 or early 2010.

To date the only operating 16” RO membrane is in Singapore. The facility, engineered, procured and constructed (EPC) by US-based CH2M HILL, is the world’s first full-scale seawater desalination plant to use large-diameter Sea Water RO (SWRO) membrane technology. Built at a cost of approximately S$20 million, the project was successfully delivered by CH2M HILL as the EPC contractor, over a fast-track schedule of 18 months, from contract signing to plant handover. The use of 16-inch diameter membrane elements instead of the current industry standard of 8-inch diameter – results in significant capital and operating cost savings to PowerSeraya.

The 10,000-m3/day (2.64-MGD) SWRO desalination plant produces two customized grades of water: High-grade service water used for power production, and potable drinking water. CH2M HILL achieved significant cost savings for the project by utilizing the power plant’s existing infrastructure of cooling water intake, discharge outfalls and on-site storage tanks. The project was completed without a single lost time incident (LTI) over the 300,000 man hours required for construction, commissioning, and operation.

The desalination plant features granular media filtration pretreatment and a two-pass SWRO system. CH2M HILL selected a SWRO technology supplied by GrahamTek for the new SWRO desalination plant. The GrahamTek technology uses 16-inch spiral wound reverse osmosis elements incorporated with an integrated flow distributor and electromagnetic field device. Advantages of this technology include a reduced footprint and capability to operate at higher

flow rates of about 1.5 times that of conventional SWRO technologies — resulting in lower capital and operating costs.

Since the number of manufacturers who are producing 16” RO membranes are currently limited and the daily production of water required to meet the demand of the cooling towers is relatively small, it is recommended that planning of advanced treatment facilities be based on use of 8-inch elements.

4.8.1.5 CIP System

Chemical cleaning may be required about once every 6 months. Membrane cleaning detergents that are certified under American National Standards Institute/National Science Foundation Standard 60 for drinking water would be used. The chemicals should not contain oxidants or other substances harmful to the membranes.

4.8.1.6 Post-treatment-Decarbonation

Decarbonation may or may not be required, depending on the alkalinity of the feed water. If feed water contains high levels of bicarbonate, a decarbonation process can be used remove excess carbon dioxide by stripping through a packed tower using counter-current forced-draft air. CO2 removal can increase the pH and, therefore, help stabilize the product water.

4.8.1.7 Advanced Oxidation Process

The RO permeate would be passed through an ultraviolet advanced oxidation (UV/AOX) process to destroy any contaminants of concern that pass through the RO membranes. Experience in southern California has shown that NDMA and 1,4-dioxane (both carcinogenic compounds) can pass through RO. Therefore, UV/AOX as has been used to provide additional treatment.

The UV/AOX process can disinfect the water and remove micro pollutants.

The key design criteria for a UV/AOX process include transmittance, flow rate, lamp sleeve fouling/lamp aging factors, the target pathogen inactivation level, the target reduction levels for micro pollutants (NDMA and 1,4-dioxane), and the level of redundancy. Additional criteria to consider in establishing design criteria include the potential for off-specification operation and the potential impacts of changes in source water quality.

The main components of UV/AOX systems are the UV light system and a delivery system for hydrogen peroxide. Various reactor configurations and lamp arrangements have been developed to optimize disinfection and oxidation performance. UV lamps are classified as low-pressure and medium-pressure lamps. Low-pressure lamps are more efficient (on a disinfection-per-watt of input power basis) than medium-pressure lamps. However, the UV output per lamp in low-pressure systems is less, so more lamps are required. Typically, the optimum lamp configuration for a given project depends on a detailed technology evaluation that considers site- and water-specific factors.

The two primary UV lamp technologies are low-pressure/high-output and medium-pressure. Selection of a preferred alternative cannot be made until additional water quality data are obtained.

4.9 Summary of Treatment Technology Screening

Based on the initial screening evaluation presented above, the treatment technologies presented in Table 4-4 were retained for further analysis. The remaining technologies were evaluated using a more rigorous benefit-to-cost ratio analysis to determine which combination of treatment processes offers the best value.

Table 4-4
Summary of Treatment Technologies Retained for Evaluation after Initial Screening

Technology	Comments
Pretreatment	
Rapid-mix coagulation and flocculation before direct filtration	Not suitable as pretreatment for cloth media filtration
Rapid-mix coagulation, flocculation, and lamella plate/ tube clarification before filtration	
High-rate ballasted clarification (Actiflo®) before filtration	
Filtration	
Continuous Backwash, Upflow Sand Media Filtration	If clarification were not included in pretreatment, 80-inch-deep filter beds would be required. 40-inch beds may be feasible with clarification ahead of filtration
Cloth Media Filtration	Pretreatment with clarification will be required
Immersed Micro/Ultra filtration	MF/UF can be used to produce disinfected tertiary recycled water; it can also pretreatment ahead of Reverse Osmosis treatment
Disinfection	
Imported Bulk Sodium Hypochlorite Solution	Disinfection with sodium hypochlorite would require construction of chlorine contact tanks.
On-site Generation of Low-strength Sodium Hypochlorite Solution	Disinfection with sodium hypochlorite would require construction of chlorine contact tanks.
Low-pressure/High-intensity Ultraviolet Irradiation	Only those systems that have been certified under the 2003 NWRI/AWWA-RF guidelines are included.
Advanced Treatment	
Reverse Osmosis Followed by UV and Oxidation methods as necessary	Advanced treatment would be based on similar installation to be further investigated during CER phase of the project

4.10 Benefits and Costs of Treatment Technologies

Benefit-to-cost ratio analyses were performed to identify the treatment technologies that offer the best value for producing "disinfected tertiary recycled water" to serve irrigation, commercial use such as toilet and urinal flushing, and UAL co-generation and Airport Central cooling towers.

Since there is only one viable alternative for advanced treatment to supply recycle water UAL co-generation and Airport Central cooling towers. It is assumed that advanced treatment consisting of MF/UF, RO and advanced oxidation would be retained as an option under all scenarios.

4.10.1 Estimated Treatment Costs

4.10.1.1 Pretreatment

Table 4-5 presents a summary of estimated capital, O&M, and net annualized costs for the retained pretreatment alternatives. The listed costs only include costs associated with this treatment step; they do not include costs for downstream processes (i.e., filtration and disinfection) or ancillary facilities such as pump stations and support facilities. The listed costs are only intended to show the relative cost differences between the processes and should not be used for budgeting purposes.

The direct filtration and Actiflo alternatives have the lowest relative capital and net annualized costs by a significant margin. Therefore, these two options were retained for evaluation in combination with the remaining filtration and disinfection alternatives. The pretreatment alternatives involving lamella clarification and the High-rate blasted **Actiflo®** were eliminated from further evaluation because they have significantly higher costs but do not offer performance advantages.

Table 4-5
Estimated Relative Costs of Pretreatment Alternatives

Item	Rapid-mix coagulation before direct filtration	Rapid-mix coagulation and flocculation before direct filtration	Rapid-mix coagulation, flocculation, and lamella plate/tube clarification before filtration	High-rate ballasted clarification (Actiflo®) before filtration
Capital Cost ($ millions)	$0.1 $0.05	$0.2 $0.08	$0.35 $0.15	$0.25 $0.1
Annual O&M Cost ($ millions)				
Net Annualized Cost ($ millions)	$0.55	$0.31	$0.17	$0.12

4.10.1.2 Filtration

Table 4-6 presents a summary of estimated capital, O&M, and net annualized costs for the remaining filtration alternatives. As with the pretreatment alternatives, the listed costs for filtration technologies only include costs associated with this treatment step and should not be used for budgeting purposes.

The sand- and cloth-media filtration options would require pretreatment, which is not included in the costs listed in Table 4-6. Conversely, MF/UF would not require pretreatment. Therefore, the costs are not directly comparable unless all associated components are included.

The cloth-media filtration option has relatively high costs compared to the sand filtration options, but it offers no performance advantages. Therefore, the cloth-media filtration option was eliminated from further evaluation.

Table 4-6
Estimated Relative Costs of Filtration Alternatives

Item	40-inch Continuous Backwash Sand Filtration	80-inch Continuous Backwash Sand Filtration	Cloth-media Filtration	Immersed MF/UF
Capital Cost ($ millions)	$0.55	$0.65	$0.4	$1.15
Annual O&M Cost ($ millions)	$0.05	$0.05	$0.05	$0.2
Net Annualized Cost ($ millions)	$0.08	$0.08	$0.07	$0.26

4.10.1.3 Disinfection

Table 4-7 presents a summary of estimated capital, O&M, and net annualized costs for the remaining disinfection alternatives. As with the alternatives described above, the listed costs for disinfection technologies only include costs associated with this treatment step and should not be used for budgeting purposes.

UV irradiation following membrane filtration has the apparent lowest capital and net annualized costs. Imported bulk sodium hypochlorite solution and UV irradiation following media filtration have nearly identical costs.

Imported bulk sodium hypochlorite is disadvantageous because it would require construction of a chlorine contact tank, which would take up significant space. If the treatment facilities must be constructed underground (as seems likely at the preferred sites), additional costs that are not reflected in Table 4-7 would be incurred. Costs for constructing facilities underground would be significantly lower for the UV alternatives because they are much smaller.

Chemical handling is another disadvantage of imported bulk sodium hypochlorite solution. Chemical handling raises potential safety issues for plant workers, and it would require delivery of chemicals through city streets.

Given the relative advantages and disadvantages of the alternatives, only UV irradiation was carried forward for additional evaluation.

Table 4-7
Estimated Relative Costs of Disinfection Alternatives

Item	On-site Generation of Sodium Hypochlorite Solution	Imported Bulk Sodium Hypochlorite Solution	UV Irradiation Following Media Filtration	UV Irradiation Following Membrane Filtration
Capital Cost ($ millions)	$0.01 $0.01	$0.02 $0.01	$0.55 $0.15	$0.4 $0.15
Annual O&M Cost ($ millions)	$0.01	$0.01	$0.18	$0.17
Net Annualized Cost ($ millions)				

4.10.1.4 Combined Treatment Trains

Three possible combinations of treatment processes (trains) can be constructed from the remaining treatment technologies. These trains and their estimated costs are listed in Table 4-8. Treatment Train No. 2, which includes direct filtration using 80-inch-deep sand filters followed by UV irradiation, has the lowest estimated capital and net annualized costs.

Table 4-8
Estimated Relative Costs of Combined Treatment Train Alternatives

Item	Treatment Train No. 1	Treatment Train No. 2	Treatment Train No. 3
Pretreatment	High-rate ballasted clarification (Actiflo®) before filtration	Rapid-mix coagulation and flocculation before direct filtration	N/A
Filtration	40-inch Continuous Backwash Sand Filtration	80-inch Continuous Backwash Sand Filtration	Immersed MF/UF
Disinfection	UV Irradiation Following Media Filtration	UV Irradiation Following Media Filtration	UV Irradiation Following Membrane Filtration
Capital Cost ($ millions)	$1.4	$1.4	$1.55
Annual O&M Cost ($ millions)	$0.3	$0.28	$0.3
Net Annualized Cost ($ millions)	$0.37	$0.35	$0.38

4.10.2 Benefits of Treatment Processes

Benefit scores and benefit-to-cost ratios were calculated for the each of the three treatment train alternatives as shown in Table 4-9. The results indicate that Alternatives No. 2 has the best benefit-to-cost ratio, followed by Train No. 1 and Train No. 3.

Table 4-9
Benefit-to-Cost Ratios for Combined Treatment Train Alternatives

Process and Criteria	Weights	Treatment Train No. 1	Treatment Train No. 2	Treatment Train No. 3
Pretreatment		High-rate ballasted clarification (Actiflo) before filtration	Rapid-mix coagulation and flocculation before direct filtration	N/A
Filtration		40-inch continuous backwash sand filtration	80-inch continuous backwash sand filtration	Immersed MF/UF
Disinfection		UV irradiation following media filtration	UV irradiation following media filtration	UV irradiation following membrane filtration
Raw Benefits Scores				
Implementability	3	5	5	5
Reliability		3	3	5
Ease of Operation and Maintenance	3	4	4	4
Impacts to Cultural and Environmental Resources	3	5	5	5
Geological and Contamination Hazards	3	5	5	5
Neighborhood Impacts	3	4	4	4
Staff Training Requirements	3	4	4	3
Implementation Schedule	3	5	5	5
Weighted Benefit Scores		**105**	**105**	**108**
Benefit-to-Cost Ratio		**284**	**300**	**284**

4.11 Conclusions and Recommendations

Treatment Train No. 2 has the lowest costs and the highest benefit-to-cost ratio. Therefore, it is recommended over the other alternatives.

5.0 Evaluation of Treatment Facility Sites

5.1 Introduction

As discussed in Section 4, two treatment processes are needed to meet the SFIA requirements: tertiary treatment to meet Title 22 requirements for irrigation and dual plumbing, and advanced treatment for cooling tower water. The tertiary treatment facility will be located at the MLTP and near the secondary effluent supply. The location of the advanced treatment facility, however, can be centralized at the MLTP or be remote and near the cooling towers. The three following sites were identified and evaluated in conjunction with the centralized, MLTP location to determine the most optimum site for the advanced treatment:

1. United Airline Cogeneration Facility parking lot
2. Lot C
3. Terminal Parking Garage

5.2 Evaluation Criteria

Eight criteria were used to compare the relative benefits of the sites. The criteria encompass legal and political issues, potential environmental impacts, and other issues that could affect construction and operation of recycled water treatment facilities. The evaluation criteria are as follows:

1. **Available space:** For a site to be considered suitable there must be adequate space available on the site for constructing new treatment facilities. Some treatment processes are relatively compact (e.g., membrane filtration), whereas other processes require much more space (e.g., processes that involve conventional coagulation, flocculation, clarification, filtration, and disinfection by chlorine contact). All sites are suitable for the most compact treatment processes, but not all of the sites can accommodate the largest processes. Sites with more available space are preferable. In situations where more space is available than needed, the extra land could be used as buffer space or for mitigation measures that may be required.

2. **Ownership/management:** Sites under the direct control of SFIA are preferable. Sites that are controlled by others are less desirable because obtaining authority to use them may require negotiation of a contract and implementation of mitigation measures that could increase costs and delay the implementation schedule.

3. **Current use of site:** Construction of treatment facilities at vacant or unused sites is preferred because existing facilities and operations would not be disrupted. Construction of treatment facilities at sites that are currently in use may require relocation of the existing uses or other mitigation measures that could add cost and delay the implementation schedule.

4. **Zoning requirements:** Sites that have less restrictive zoning requirements are preferable. Construction of treatment facilities at sites with more restrictive zoning may require zoning changes or mitigation measures to comply with height and bulk restrictions, for example. Such measures may increase costs and delay the implementation schedule.

5. **Accessibility to major roads:** Easy access to major roads is desirable during construction and operation of treatment facilities. During construction, access will be required for construction equipment, material deliveries, and construction workers. During operation, access will be required for material and chemical deliveries and operations and maintenance staff. Sites located adjacent to major roads are preferable because impacts to sensitive neighbors will be minimized and the efficiency of construction and operations will increase.

6. **Seismic risk:** Sites that are within geologic hazard zones that are subject to liquefaction and landslides are less desirable. The treatment facilities can be designed to mitigate these hazards, but costs will increase and the implementation schedule may be delayed.

7. **Tsunami risk:** Sites that are within the tsunami threat zone are less desirable. The treatment facilities can be designed to mitigate this hazard, but costs will increase and the implementation schedule may be delayed.

8. **Existence of cultural and environmental resources:** Sites that do **not** have significant existing cultural and environmental resources are preferable. Use of sites that have cultural and environmental resources may require mitigation measures that would add costs and may delay the implementation schedule.

5.3 Descriptions of Sites for Advanced Treatment Facilities

Narrative descriptions of the four sites are provided below. Site maps are provided in Figure 5.1 through Figure 5.4 and a qualitative site matrix based on the evaluation criteria is presented in Table 5-1.

5.3.1 Tertiary and Advanced Treatment Facilities at MLTP

Roughly 2,500 ft², the area adjacent to the northern most SBR tank at MLTP would be most suitable for the tertiary and advanced treatment of the effluent. (See Figure 5.1) This location is near the combined effluent pump station, and would not impede current operations or future plans for a new industrial waste treatment facility. Given its proximity, power and waste utilities are easily accessible.

5.3.2 United Airline Cogeneration Facility East Parking Lot

The 720,000 ft² east parking lot, along North Access Road, is currently leased property to United Airlines. This lot is adjacent to the cogeneration plant and North Access Road. (See Figure 5.2) The advanced treatment would require approximately 625 ft² of space. Currently, the Airport does not have accessible power and waste utilities for the advance treatment process at this site.

5.3.3 Lot C

Lot C is roughly 25,000 ft² and owned by the Airport. This lot is ideally located near the terminals at the US101/SF Airport interchange. (See Figure 5.3) There are five 40,000 gallon underground storage tanks in this lot that were installed during the Airport Master Plan but never utilized. An advanced treatment unit would occupy 625 ft² of space in this lot. With power and waste utilities in the vicinity, this location is a viable option for advanced treatment.

5.3.4 Terminal Parking Lot

The proposed location for an advanced treatment unit is near the southern-most cooling tower on the 2nd level of the parking garage. (See Figure 5.4) This location offers weather protection and accessibility to maintenance staff; however, construction constraints and utility accessibility may outweigh these advantages. The proposed unit would require 625 ft² of space in the garage.

Table 5-1
Qualitative Comparison off Advanced Treatment Facility Sites

Criteria	MLTP	UA Cogen Parking Lot	Lot C	Terminal Parking Lot
Available space	Yes	Yes	Yes	Yes
Current management	SFO	SFO	SFO	SFO
Current use	Treatment Plant	Parking	None	Parking
Zoning	Open Space	Open Space	Open Space	Height Restriction
Access to major roads	Excellent	Excellent	Excellent	Good
Seismic risk	Moderate	Moderate	Moderate	High
Tsunami risk	Moderate	Low	Low	Low
Cultural and environmental resources	Limited	None	None	None

5.4 Conclusions and Recommendations

Conclusions and recommendations regarding each site are presented below that are based on the preceding evaluations.

5.4.1 Advanced Treatment at MLTP

The MLTP site is preferable to other sites because it offers:

- Currently under management by SFIA.
- Appropriate zoning already in place.
- Excellent access to major roads.
- Suitable for treating secondary effluent to produce Title 22 water and advanced treatment for cooling towers.

There are no disadvantages of the MLTP site.

Recommendation: Retain for further evaluation.

5.4.2 United Airline Cogeneration Facility Parking Lot

Beneficial attributes of the UAL Cogeneration Facility Parking Lot include:

- Adequate available space.
- Good access to major roads.

Disadvantages of the site are as follows:

- Does not have accessible power and waste utilities.
- Requires additional pumping of waste back to MLTP.
- Requires an enclosure or housing for the equipment.

Since the site requires conveying of Title 22 water for advanced treatment and is in close proximity of MLTP. The advanced treatment can be located in MLTP and advanced treated water can be delivered directly to UAL cogeneration cooling tower.

Recommendation: Do not carry forward for further evaluation.

5.4.3 Lot C

Beneficial attributes of the Lot C site include:

- Adequate available space.
- Currently under management by SFIA.
- Excellent access to major roads.
- Can utilize existing underground storage tanks.
- Required utilities moderately accessible.

Disadvantages of the site are as follows:

- Requires an enclosure or housing for the equipment.

Recommendation: Retain for further evaluation.

5.4.4 Terminal Parking Lot

Beneficial attributes of the Lot C site include:

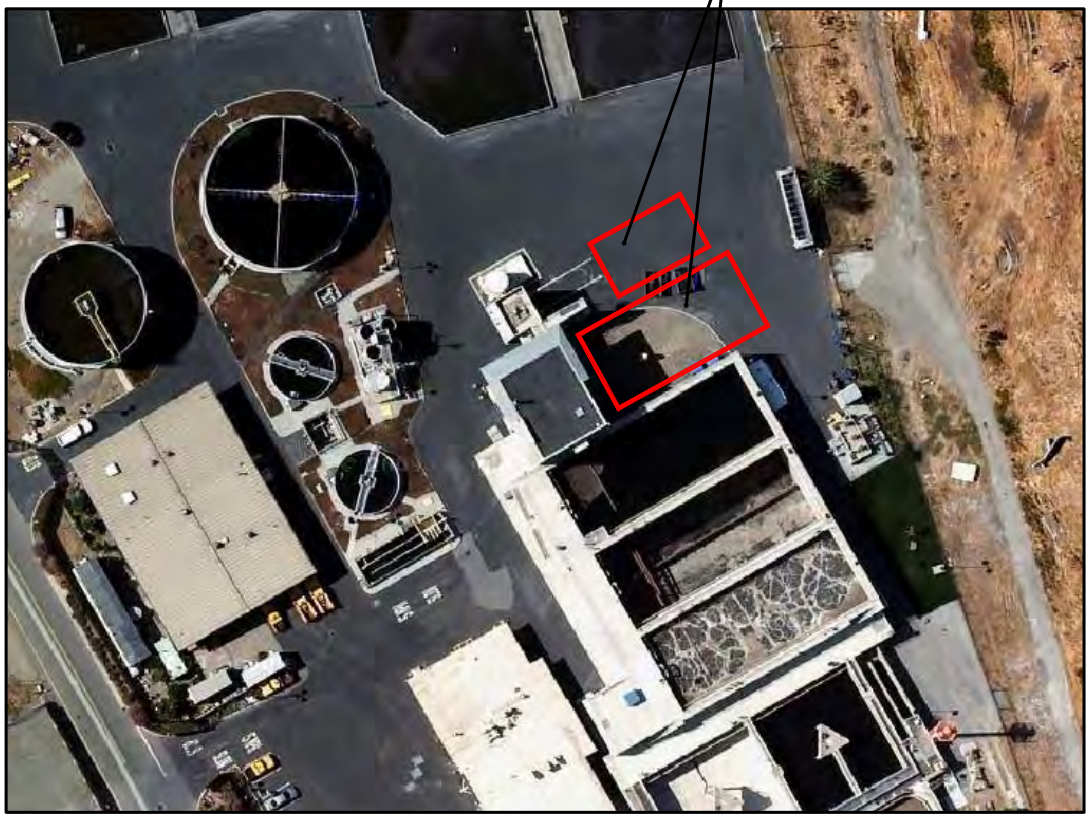
- Good access to major roads.
- Weather protection.
- Close accessibility to maintenance staff.

Disadvantages of the site are as follows:

- Height constraint during construction and installation of equipment.
- Not zoned for construction and installation of treatment facilities.

Recommendation: Do not carry forward for further evaluation.

SITE MAP



TERTIARY AND ADVANCED TREATMENT FACILITIES IN MLTP

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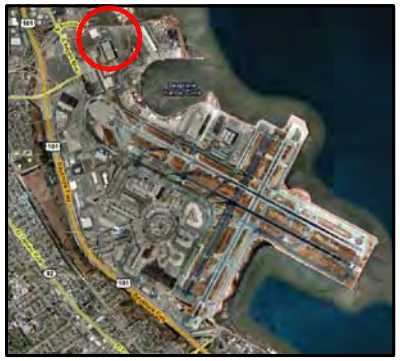

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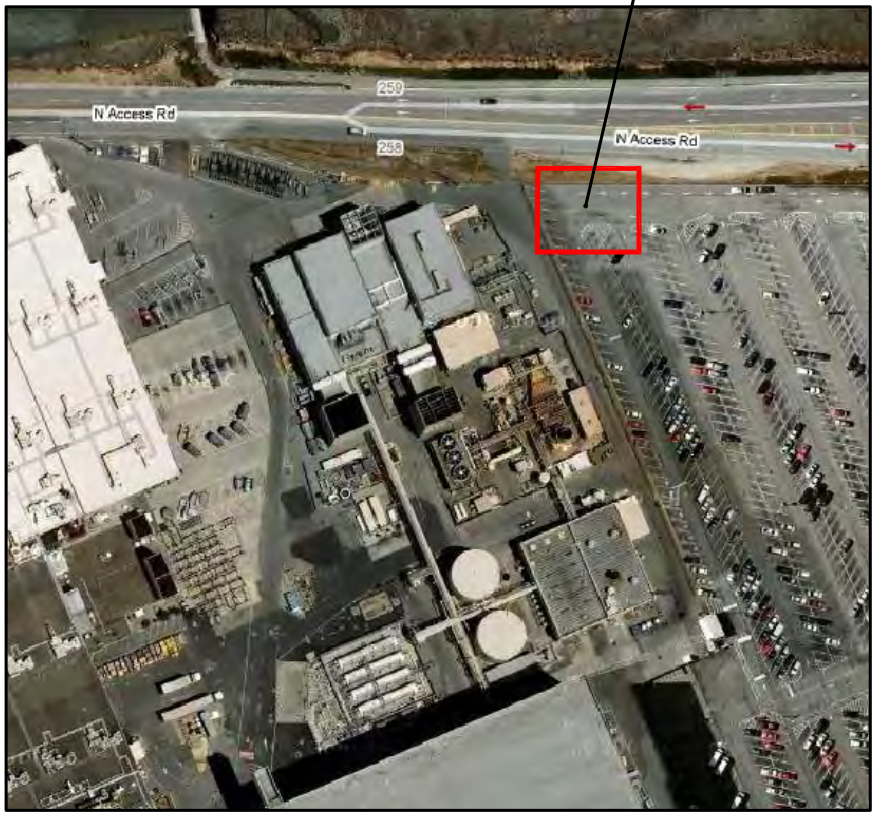

UA COGEN PARKING LOT

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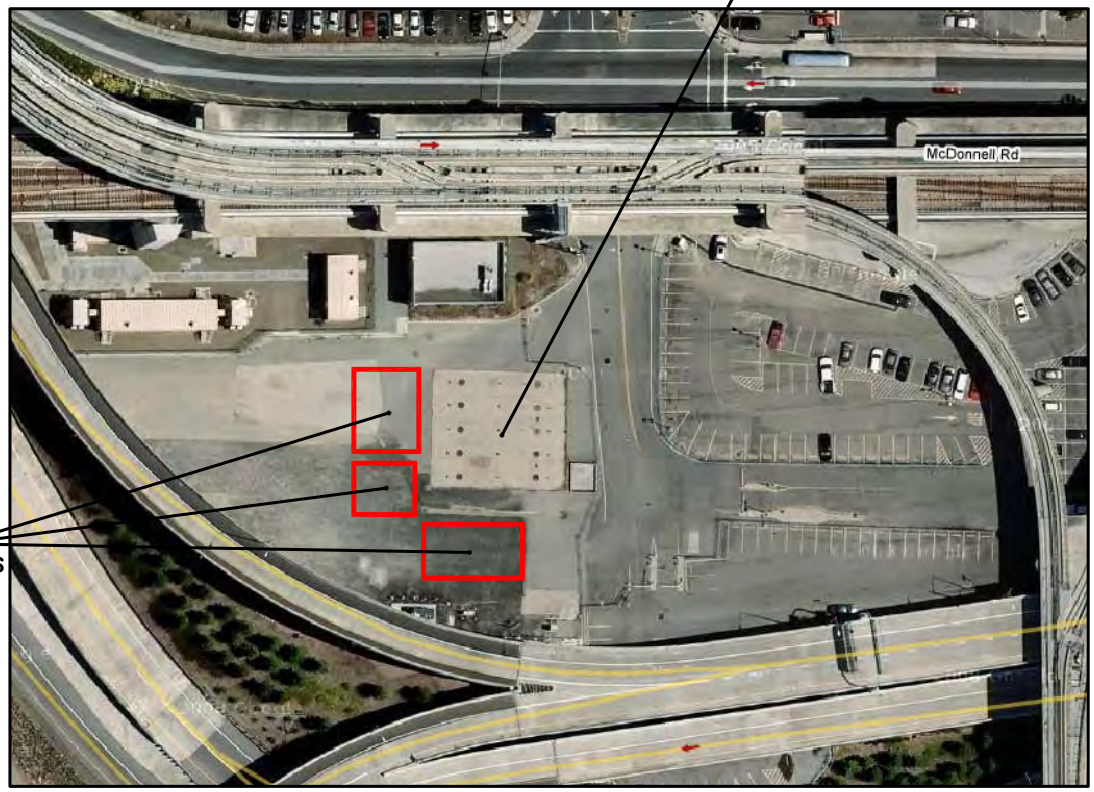

LOT C

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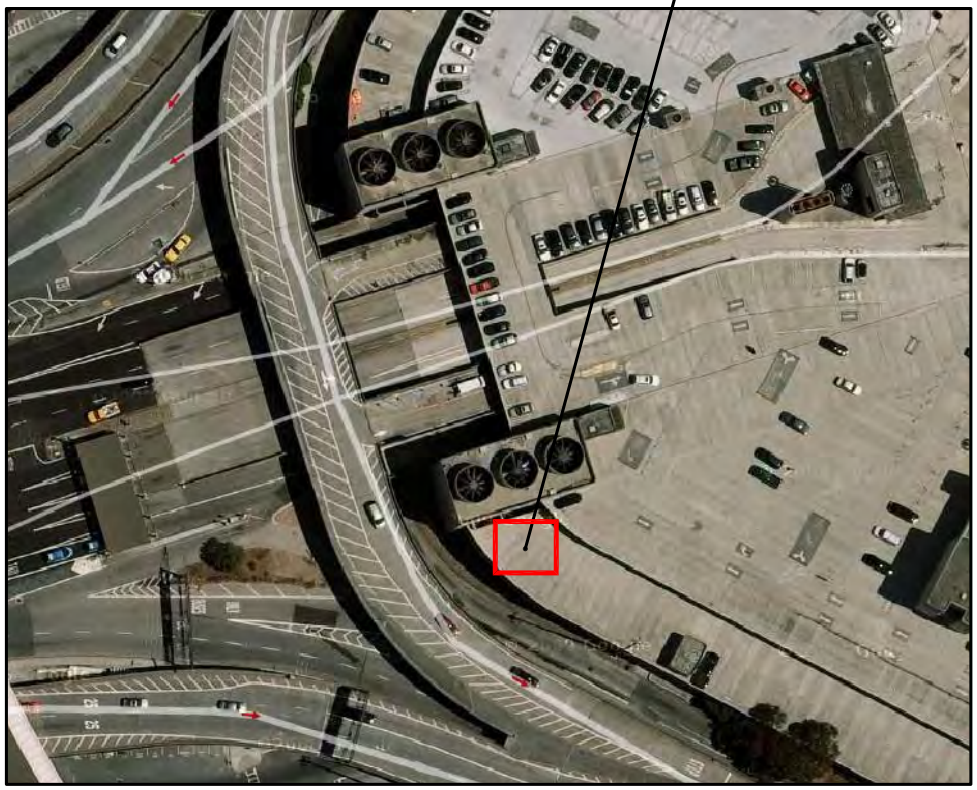

TERMINAL PARKING GARAGE

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6.0 Distribution System Evaluation

6.1 Introduction

A distribution system will be required to convey recycled water from the treatment facilities to the users. Major components of distribution systems would include pump stations, storage tanks, and pipelines.

Four alternatives for distributing recycled water to potential users were developed and evaluated. The alternatives are configured to serve potential users from the two preferred sites for new advanced treatment facilities and irrigation supply system.

This section includes comparisons of the four alternatives for the distribution system and preliminary design criteria for pipes, pump stations, and storage tanks.

6.2 Regulations and Guidelines for Distribution System

The following regulations and guidelines were referenced in preparing the analysis of distribution alternatives:

- *California Health Laws Related to Recycled Water* (aka, "The Purple Book") (California Department of Health Services 2001)
- *Guidelines for Water Reuse* (USEPA 2004)
- *Guidelines for Distribution of Nonpotable Water*, California-Nevada Section (AWWA 1992)

These documents contain recommendations regarding pressure, minimum burial depth, separation distances from other utilities (especially potable water), pipe identification, isolation valves, air relief and vacuum valves, and blowoff assemblies. The documents also contain recommendations for seasonal storage, operational storage, and backup water supplies, as well as considerations for retrofitting customers' facilities to use recycled water.

It is recommended that the regulatory requirements and guidelines contained in the documents be used throughout design, construction, startup, and operation of the facilities. In addition, *Guidelines for the On-Site Retrofit of Facilities Using Disinfected Tertiary Recycled Water*, California-Nevada Section AWWA (1999) should be referenced when planning and designing retrofits of facilities to use recycled water.

6.3 Configurations of Distribution Systems

The recycled water demands and locations of end users and treatment facilities were used to evaluate options for configuring the distribution systems to serve the end users.

6.3.1 Configuration Alternatives

Four separate alternatives for configuring the distribution system were evaluated to identify the alternative that offers the most benefit for the lowest cost. The four alternatives include:

- Centralized advanced treatment at MLTP.
- Decentralized advanced treatment at MLTP and Lot C.
- Centralized irrigation supply at Lot C.
- Decentralized irrigation supply at Lot C and San Bruno Site.

"Centralized" alternatives include both tertiary and advanced treatment facilities located at the MLTP site, and irrigation supply at Lot C. Conversely, "decentralized" alternatives entail construction of separate advanced treatment facilities and irrigation supply at two of the preferred sites.

The four alternatives are depicted in Figures 6.1 through 6.4. The pipeline routes are based on both directional drilling and open trench construction methods. The figures also show proposed locations of storage tanks and pump stations that are described later.

Centralized Advanced Treatment at MLTP - In this scenario, a single advanced treatment facility would be sized to meet the demands for both the United Airline Cogeneration and Main Terminal cooling towers to be located at the MLTP near the tertiary treatment facility. Advanced treated water would be stored in a new storage tank with two dedicated pumps delivering treated water to the United Airline Cogeneration cooling tower and Main Terminal cooling towers.

Decentralized Advanced Treatment - In the decentralized scenario, two advanced treatment facilities will be constructed in the distribution system. One would be located at MLTP and sized to meet the demand for the United Airline Cogeneration cooling tower. A new storage tank with a dedicated pumping system would deliver advanced treated recycled water to the United Airline Cogeneration cooling tower. The other facility will be located in Lot C and sized to meet the Main Terminal cooling towers' demands. A similar delivery system will be constructed in Lot C to deliver advanced treated recycled water to the Main Terminal cooling towers.

Centralized Irrigation Supply at Lot C - Disinfected tertiary recycled water will be distributed from a primary storage tank at the MLTP to the existing Lot C storage tanks. The Lot C storage tank and hydro pneumatic pumping system will be sized to supply recycled water for all the landscape irrigation around the San Bruno area and Lot C area as well as dual plumbing facilities in the terminals. The irrigation supply system near Lot C will require control valves and pressure reducing valves to offset the high discharge pressure from the pumping system.

Decentralized Irrigation Supply - In this scenario, disinfected tertiary recycled water will be distributed from MLTP to two, individual irrigation supply systems: San Bruno site and Lot C. The San Bruno irrigation system will be sized to supply recycled water for landscape irrigation around the San Bruno site as shown in Fig. 6.4 and, likewise, the Lot C irrigation system will be sized to supply recycled water for landscape irrigation around Lot C area as well as dual plumbing facilities in the terminals. The pumping system at Lot C will be smaller and pressure reducing valves will not be required since the pumping system discharge pressure will be designed to meet the pressure requirement at Lot C area only.

6.3.2 Pipeline Alignment Alternatives

Three alignments were initially proposed for the routing of the main distribution. However, selection of the final pipeline alignment depends upon the configuration selected in the preceding section. Alignments were evaluated for their feasibility, design and construction constraints, and overall operation of the system.

The criteria used for evaluating and selecting an alignment were as follows:

1. shortest distance from RWF to end-user point,
2. most direct route that avoids critical areas of the AOA as best as possible
3. lowest construction cost using materials that provide long term reliability of the distribution system

Of the three alternatives, two were rejected because they failed to meet the listed criteria. One alternative proposed that the distribution main be routed along city right-of-ways (North Access Road/McDonnell Road). Although this alignment circumvents the airfield, it fails to meet the criteria of being the shortest route between the MLTP and Lot C. Further, this alternative did not offer a distinct advantage when compared to other options. The second alternative proposed to install the distribution main under active taxiways and runways. While this alternative represented the shortest possible distance between the MLTP and Lot C, it failed to meet the second criteria; namely, avoiding critical AOA infrastructure. Moreover, installing a main under the critical areas of the AOA would require that a pipe be deep enough to not disturb/impact the taxiway/runway base. Pipe failure in this instance could have catastrophic political and financial consequences.

The remaining viable alignment that met all the criteria is shown in Figure 6.2. This alignment represents the shortest route between the MLTP and Lot C, and avoids all major taxiways and runways.

6.3.3 Pipeline Design Criteria

Ideally, the system would operate under gravity flow. However, the pipes will be designed to operate in forced flow condition if pumps are needed to meet future requirements. Distribution pipes will be sized to maintain flow velocities below 7 ft/s while maintaining maximum head loss limits for future system expansion. The minimum pipe diameter within the system is 4 inches. Pipe diameters for each of the alternatives are shown in conjunction with the proposed alignments (section 6.3.2) in Figures 6.1 through 6.4.

6.3.4 Pipeline Construction Methods

Four methods of construction were identified for the installation of the distribution pipes: namely, open trench, directional drilling, micro-tunneling, and jack and bore. The latter three are categorized as trenchless technology. Trenchless technology is desirable because it is the least invasive method of installation; allowing roadways and taxiways to potentially remain active while utilities are being installed. This method, however, requires precision utility

locating and presents a greater risk when crossing existing utilities. Although, open trenching is the most invasive method of installation, it does expose existing utilities in the trench, thus reducing the risk of compromising an existing utility.

Of the four identified methods above, the micro-tunneling and jack-and-bore methods were rejected because they require a straight alignment and are more ideal for gravity line applications where constant slopes need to be maintained. Conversely, directional drilling is more dynamic during installation and offers greater flexibility. Generally, pipes can be installed in longer "pulls" with directional drilling which reduces mobilizations. Typical "pulls" are 500 feet in length as opposed to 100 – 200 feet with micro-tunneling or jack and bore. Directional drilling allows the operator to change vertical and horizontal directions when installing the pipe. (Plastic pipe presents a specified bending radius which offer distinct installation advantages.) In contrast, open trench installations are labor intensive but are inherently safer in comparison to directional drilling.

Ideally, the distribution system will be constructed using both methods. In areas where the distribution pipes cross critical pipes (force mains, gas lines, aviation fuel lines) or when pipe layout areas are not feasible, open trenching may be required. Directional drilling though would be the preferred method based on lower per-foot costs.

6.3.5 Pump Stations

Regardless of the configuration selected in section 6.3.1, multiple pumps stations would be required to deliver water from the MLTP to the end users. A primary pump station, located at the MLTP, would distribute treated water to the storage facilities in Lot C. A smaller pump stations would then deliver water from the Lot C facilities to selected end users (irrigation, cooling towers, etc.) that do not require a constant pressure. Since the dual plumbing facilities in the terminal require a constant operating pressure, a hydro-pneumatic tank would be constructed in Lot C.

Estimated flow and discharge pressure capacities for the pump stations required for each alternative are listed in Table 6-1. These estimates should be refined during later phases of implementation.

Table 6-1
Estimated Requirements for Pump Stations

Item	Estimated Flow (gpm) and Pressure (psig) at Peak Demand			
	Centralized advanced treatment at MLTP	Decentralized advanced treatment at MLTP and Lot C	Centralized Irrigation Supply at Lot C	Decentralized Irrigation Supply at Lot C and San Bruno site
Secondary Effluent Pump Station	525 @ 35	525 @ 35	525 @ 35	525 @ 35
Primary Pump Stations	--	425 @ 50	425 @ 50	425 @ 50
San Bruno Pump Station	--	--	--	300 @ 85
Lot C Pump Station	--	--	750 @ 115	450 @ 95
UAL CT Pump Station	120 @ 95	120 @ 95	--	--
Terminal CT Pump Station	100 @ 115	100 @ 95	--	--

6.3.6 Storage Tanks

A main storage tank at the tertiary treatment facility will be required to store the treated Title 22 water for distribution. In addition, storage tanks will also be required for advanced treated water for distribution to the cooling towers.

At Lot C, the existing five 40,000 gallon underground tanks will be utilized. Depending on the configuration alternative selected, the tanks can be modified to maximize the storage capacity of Title 22 and/or advanced treated water.

Storage sizing will be performed during the design phase to meet ADMM demand based on the assumed diurnal use patterns using a mass-balance equation with a 1-hour time step. The equation to be used is as follows:

$$V_{n+1} = V_n + Inflow - Outflow$$

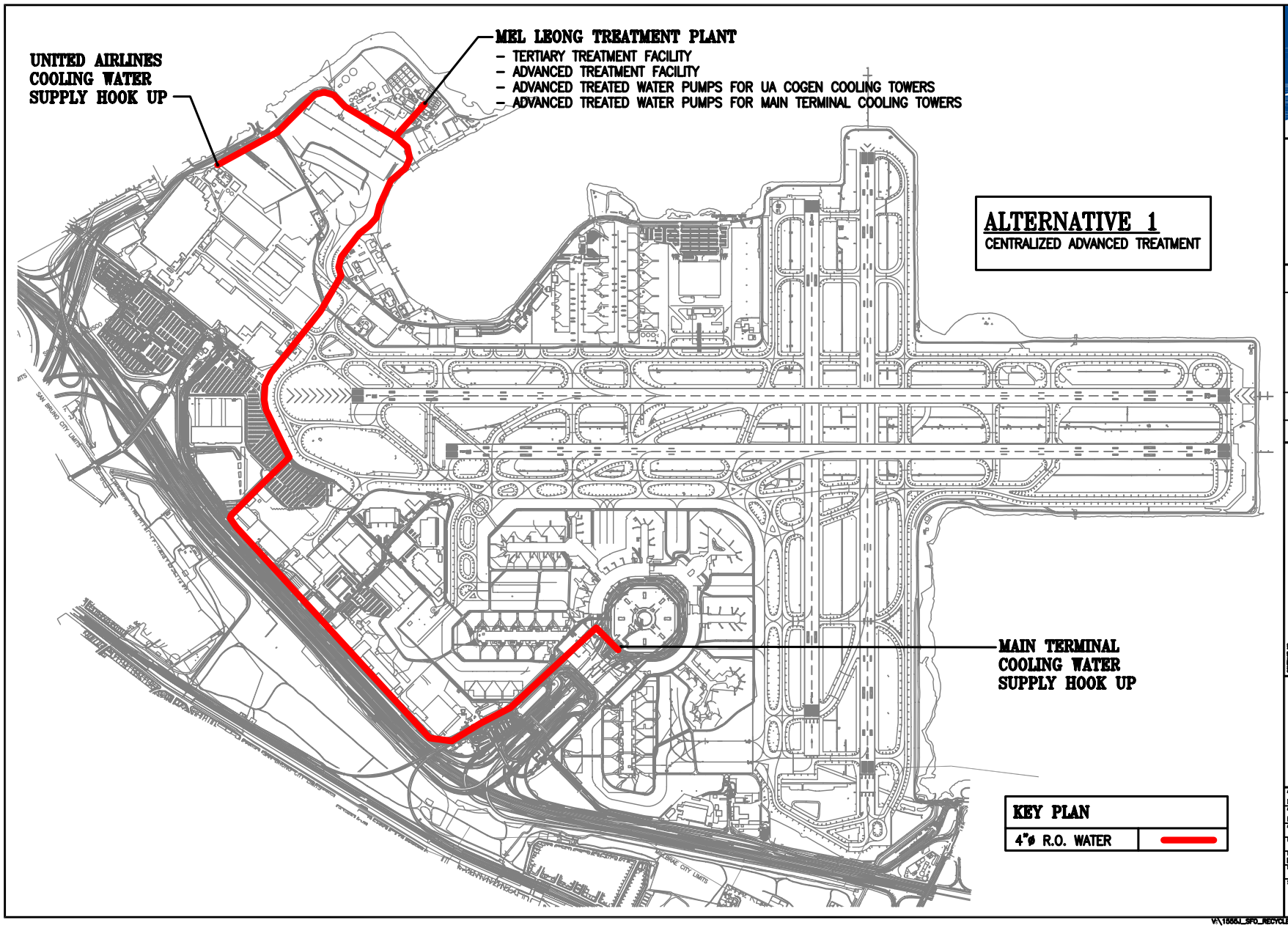
Where:

V_n = volume of water in the storage at time *n*.

V_{n+1} = volume of water in the storage at time *n + 1 hour*.

Inflow = amount of water added to the storage tank during the 1-hour time step.

Outflow = amount of water removed from the storage to meet demands during the 1-hour time step.



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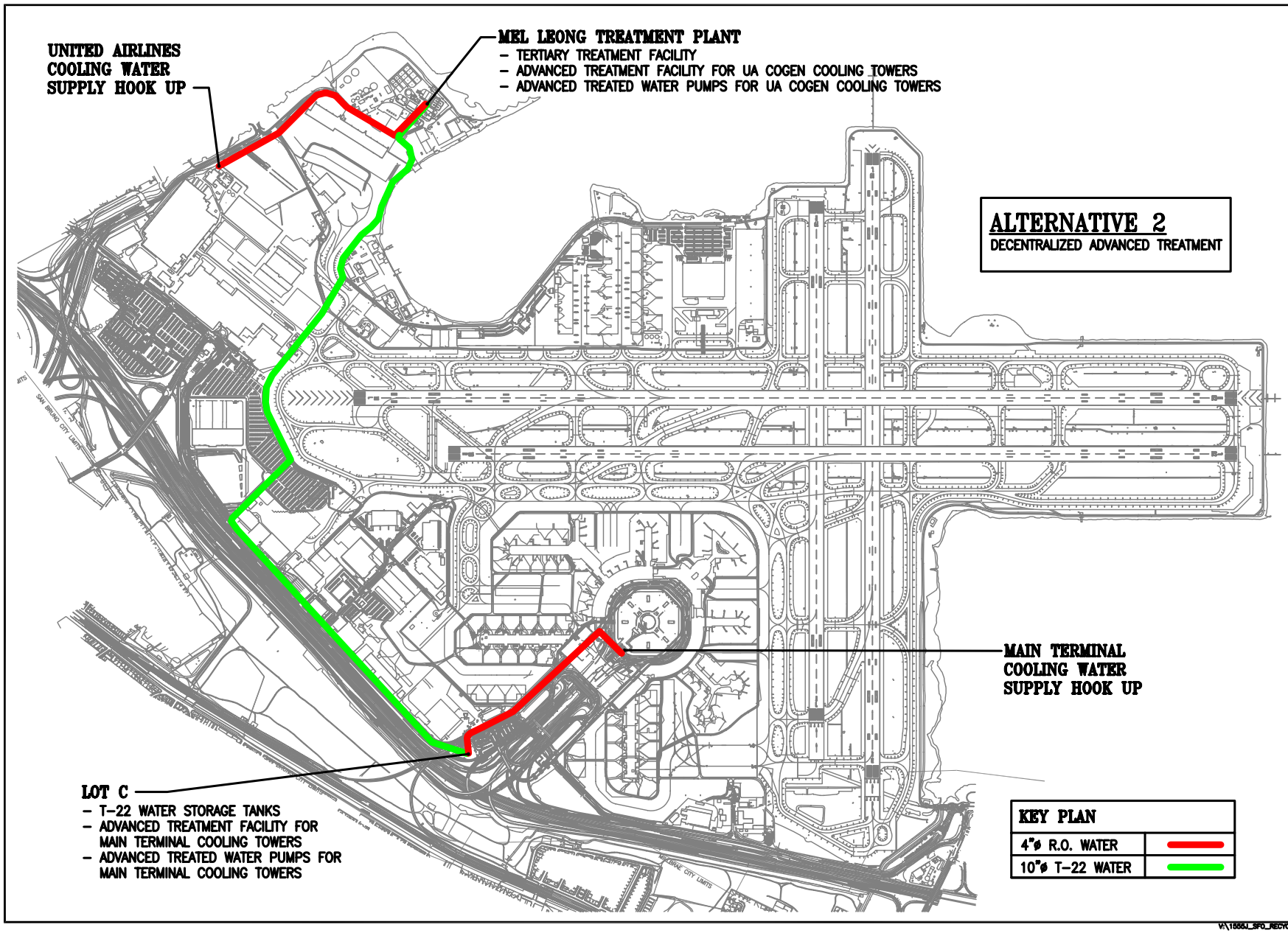

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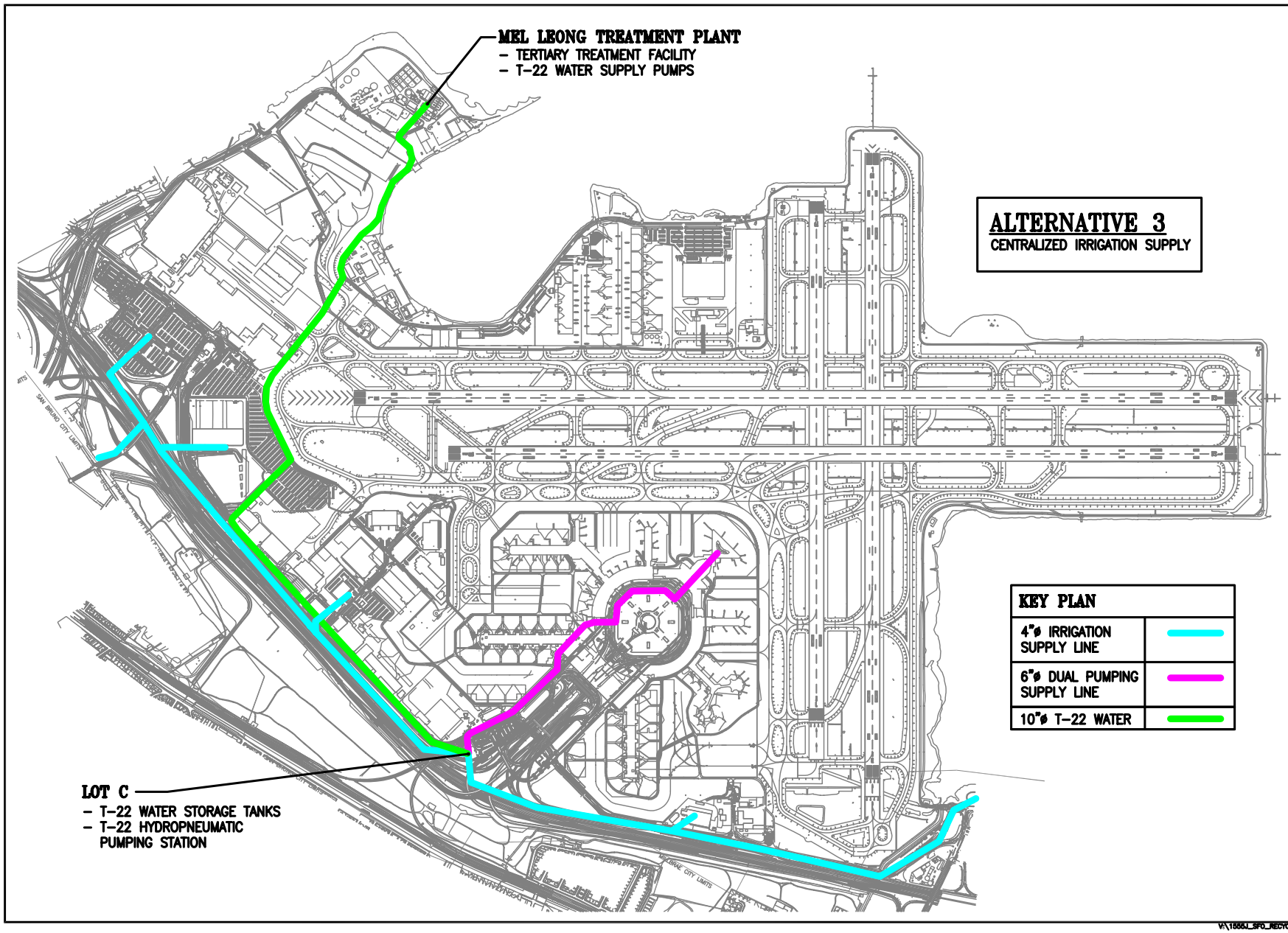

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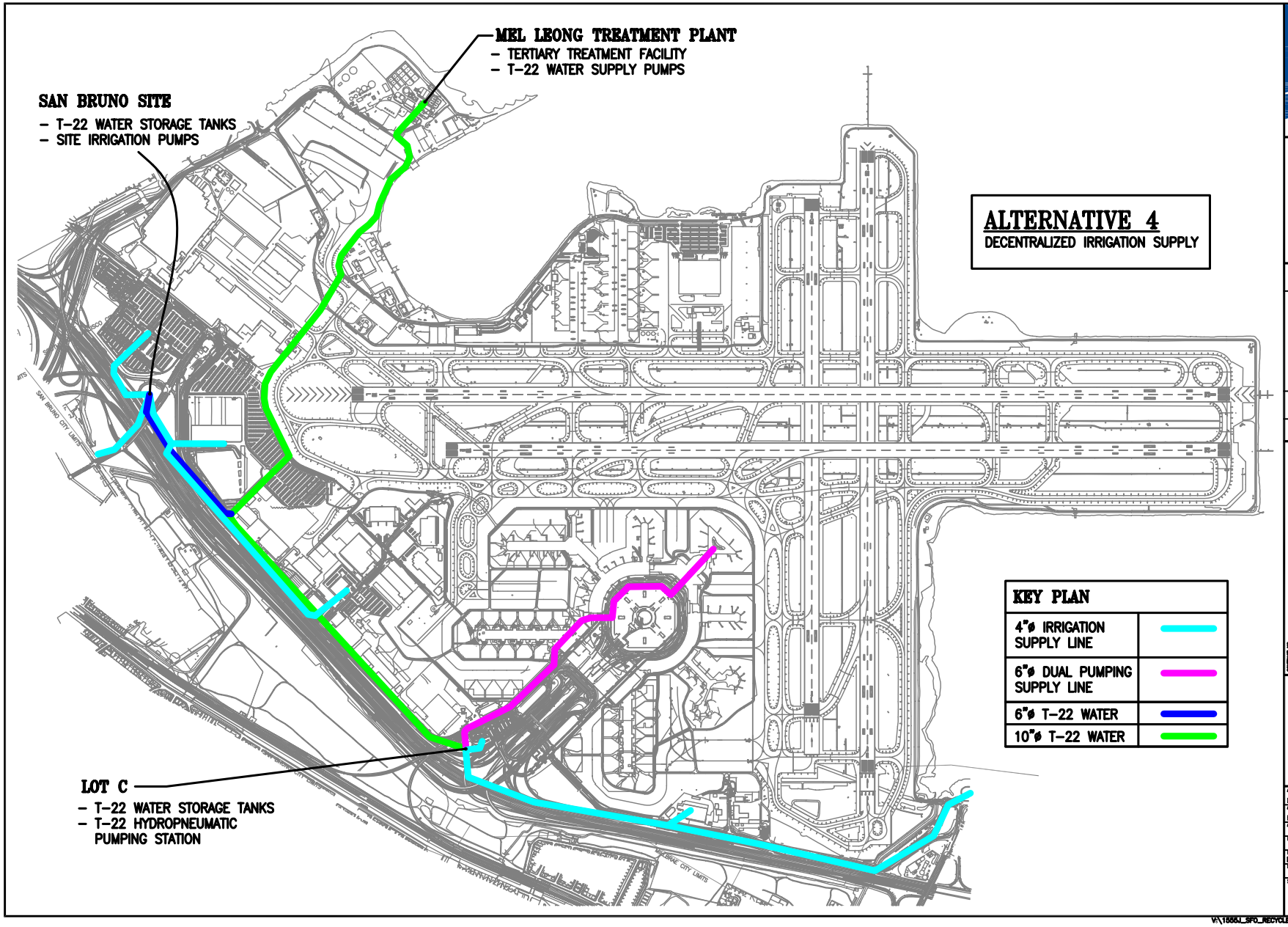

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7.0 Development of Combined Scenarios

7.1 Introduction

The following preferred alternatives were identified in previous sections of this report for the treatment sites, treatment technologies, and distribution system:

- Preferred Treatment Sites
  - MLTP
  - Lot C
- Preferred Treatment Technologies
  - "Disinfected tertiary recycled water": Direct filtration using rapid-mix coagulation, 60-inch-deep continuous backwash sand filtration, and disinfection by UV irradiation
  - Advanced treatment to cooling towers: Pressurized microfiltration/ultrafiltration and reverse osmosis
- Preferred Distribution Alternatives
  - Centralized advanced treatment at MLTP.
  - Decentralized advanced treatment at MLTP and Lot C.
  - Centralized irrigation supply at Lot C.
  - Decentralized irrigation supply at Lot C and San Bruno Site.

These alternatives were combined into the four different scenarios listed in Table 7-1. The difference between each scenario is the placement of the advanced treatment facilities and irrigation supply system. The proposed treatment technology, however, remains constant for each scenario.

Scenario 1: Centralized treatment with centralized irrigation supply.
Scenario 2: Centralized treatment with decentralized irrigation supply.
Scenario 3: Decentralized treatment with centralized irrigation supply.
Scenario 4: Decentralized treatment with decentralized irrigation supply.

Table 7-1
Combined Treatment and Distribution Scenarios

Item	Scenario No.1	Scenario No.2	Scenario No.3	Scenario No.4
Location of "disinfected tertiary recycled water" treatment facilities	MLTP	MLTP	MLTP	MLTP
Location of advanced treatment facilities	MLTP	MLTP	MLTP and Lot C	MLTP and Lot C
Tertiary Treatment Technology	Direct filtration using rapid-mix coagulation, 60-inch-deep continuous backwash sand filtration, and disinfection by UV irradiation	Direct filtration using rapid-mix coagulation, 60-inch-deep continuous backwash sand filtration, and disinfection by UV irradiation	Direct filtration using rapid-mix coagulation, 60-inch-deep continuous backwash sand filtration, and disinfection by UV irradiation	Direct filtration using rapid-mix coagulation, 60-inch-deep continuous backwash sand filtration, and disinfection by UV irradiation
Advanced Treatment Technology	Pressurized MF/UF and reverse osmosis	Pressurized MF/UF and reverse osmosis	Pressurized MF/UF and reverse osmosis	Pressurized MF/UF and reverse osmosis
Distribution System	Irrigation supply system at Lot C	Irrigation supply system at Lot C and San Bruno Site	Irrigation supply system at Lot C	Irrigation supply system at Lot C and San Bruno Site

7.2 Configuration of Combined Scenarios

Figures 7.1 through 7.4 show the proposed configurations of the four scenarios, and Figures 7.5 through 7.8 show schematic process flow diagrams. Drawings showing preliminary layouts of the treatment facilities and irrigation supply system on different sites are provided in Figures 7.9 through 7.11. (All figures for this section are presented at the end of this section.)

7.3 Estimated Costs

Estimates of costs for construction and operation of the recycled water system were prepared for each of the four scenarios as shown in Table 7-6.

7.3.1 Basis of Cost Estimates

Assumptions and details of the cost estimates are contained in the appendices of this report.

The cost estimates were prepared using industry standard cost estimating practices, references, costs of similar projects, and material quotes from vendors. The costs presented herein are considered a *Class 4 Estimate*, as defined by AACE International (the Association for the Advancement of Cost Engineering) for construction cost estimating. Class 4 cost estimates are generally prepared during preliminary planning to form the basis for the project authorization and/or funding. The typical expected accuracy range for this class of estimate is -15 to -30 percent on the low side and +20 to +50 percent on the high side.

The opinions of cost shown and any resulting conclusions on project financial or economic feasibility or funding requirements have been prepared for guidance in project evaluation and implementation from the information available at the time the opinion was prepared. The final project costs will depend on actual labor and material costs, competitive market conditions, actual site conditions, final project scope, implementation schedule, continuity of personnel and engineering, and other variable factors. As a result, the final project costs may vary from the opinions of cost presented herein. Because of these factors, project feasibility, benefit/cost ratios, risks, and funding needs must be carefully reviewed prior to making specific financial decisions or establishing project budgets to help ensure proper project evaluation and adequate funding.

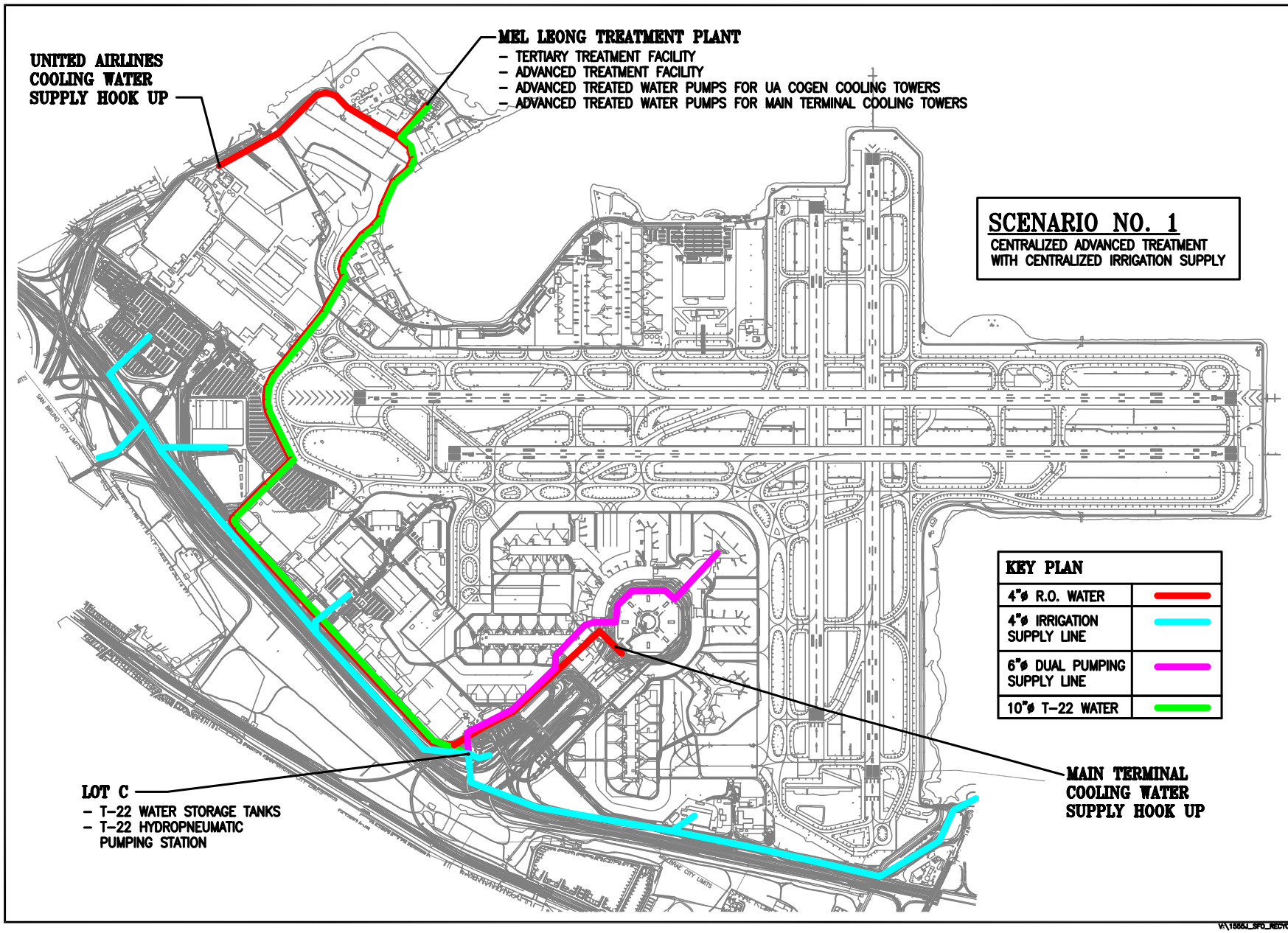
7.3.2 Summary of Estimated Costs

As indicated in Table 7-2, Scenarios 1 and 2 have higher pipeline costs with the construction of a dedicated force main to deliver advanced treated water from MLTP to the terminal in addition to the Title 22 water distribution main from MLTP to Lot C. With Scenarios 3 and 4, the advanced treated water force main will only be from Lot C to the terminal, but the costs for advanced treatment facility are higher with two treatment facilities versus one for Scenarios 1 and 2. Overall Scenario 3 has the lowest Program Cost and Scenario 2 has the highest Program Cost and overall Net Annualized Costs are within plus/minus 10% of each other.

Table 7-2
Summary of Estimated Capital, O&M and Lifecycle Costs of Combined Scenarios

Item	Scenario No.1	Scenario No.2	Scenario No.3	Scenario No.4
Tertiary Treatment Facilities				
Pretreatment	$100,000	$100,000	$100,000	$100,000
Continuous Backwash Sand Filtration	$525,000	$525,000	$525,000	$525,000
UV Disinfection	$415,000	$415,000	$415,000	$415,000
Chemical System	$ 75,000	$ 75,000	$ 75,000	$ 75,000
Structural Foundation and Piles	$380,000	$380,000	$380,000	$380,000
Support Facilities				
Secondary Effluent Pump Station	$71,000	$71,000	$71,000	$71,000
Secondary Effluent Force Main	$22,500	$22,500	$22,500	$22,500
Modification of (E) Effluent Pump Station	$25,000	$25,000	$25,000	$25,000
Title 22 Distribution System				
Pump Stations				
Primary Pump Station	$135,000	$135,000	$135,000	$135,000
Lot C Pump Station w/Hydro Tanks	$250,000	$250,000	$250,000	$250,000
San Bruno Pump Station	--	$125,000	--	$100,000
Storage Tanks				
Main Storage Tank	$700,000	$700,000	$700,000	$700,000
Lot C Storage Tank Retrofit	$500,000	$500,000	$750,000	$500,000
San Bruno Storage Tank	--	$500,000	--	$500,000
Advanced Treatment Facilities				
UF/RO	$585,000	$585,000	$875,000	$875,000
UA Pump Station	$ 95,000	$ 95,000	$ 95,000	$ 95,000
Terminal Pump Station	$110,000	$110,000	$ 95,000	$ 95,000
O&M Building at Lot C	--	--	$175,000	$175,000

Electrical and I&C Work	$775,000	$775,000	$775,000	$775,000
Mechanical and Process Piping	$500,000	$500,000	$500,000	$500,000
Structural and Site Work	$250,000	$250,000	$250,000	$250,000
Subtotal	$5,513,500	$6,063,500	$6,083,500	$6,633,500
Mob/Demob/Bonds/Insurance	$250,000	$250,000	$250,000	$250,000
Allowances	$150,000	$150,000	$150,000	$150,000
Subtotal	$5,913,500	$6,463,500	$6,483,500	$7,033,500
15% Contractor Markup	$887,000	$969,525	$972,525	$1,005,000
15% Contingency	$887,000	$969,525	$972,525	$1,005,000
Subtotal Construction Cost	$7,687,500	$8,402,550	$8,428,550	$9,143,500
Pipelines				
80% D. Drilling + 20% Trench	$5,428,000	$5,338,000	$3,912,000	$3,872,000
Subtotal Construction Cost	$13,115,000	$13,791,000	$12,340,000	$13,015,000
7.5% Escal/Local Adjust Factor	$ 985,000	$ 1,035,000	$ 926,000	$ 976,000
Total Construction Cost	$14,100,000	$14,826,000	$13,266,000	$13,992,000
25% Non Construction Cost	$ 3,525,000	$ 3,706,500	$ 3,316,000	$ 3,500,000
TOTAL PROGRAM COST	$17,625,000	$18,532,500	$16,582,000	$17,492,000
Annual O&M Costs				
Title 22 System				
Secondary effluent Pump Station	$ 25,000	$ 25,000	$ 25,000	$ 25,000
Pretreatment	$ 85,000	$ 85,000	$ 85,000	$ 85,000
Filtration	$ 65,000	$ 65,000	$ 65,000	$ 65,000
Disinfection	$110,000	$110,000	$110,000	$110,000
Pump Stations	$ 60,000	$ 90,000	$ 75,000	$ 90,000
Advanced Treatment				
Advanced Treatment Processes	$175,000	$175,000	$225,000	$225,000
Pump Stations	$ 75,000	$ 75,000	$ 75,000	$ 75,000
Total Annual O&M	$595,000	$625,000	$660,000	$675,000
Annualized Capital Cost	$1,189,000	$1,250,000	$1,118,000	$1,180,000
Net Annualized Cost	$1,784,000	$1,875,000	$1,778,000	$1,855,000



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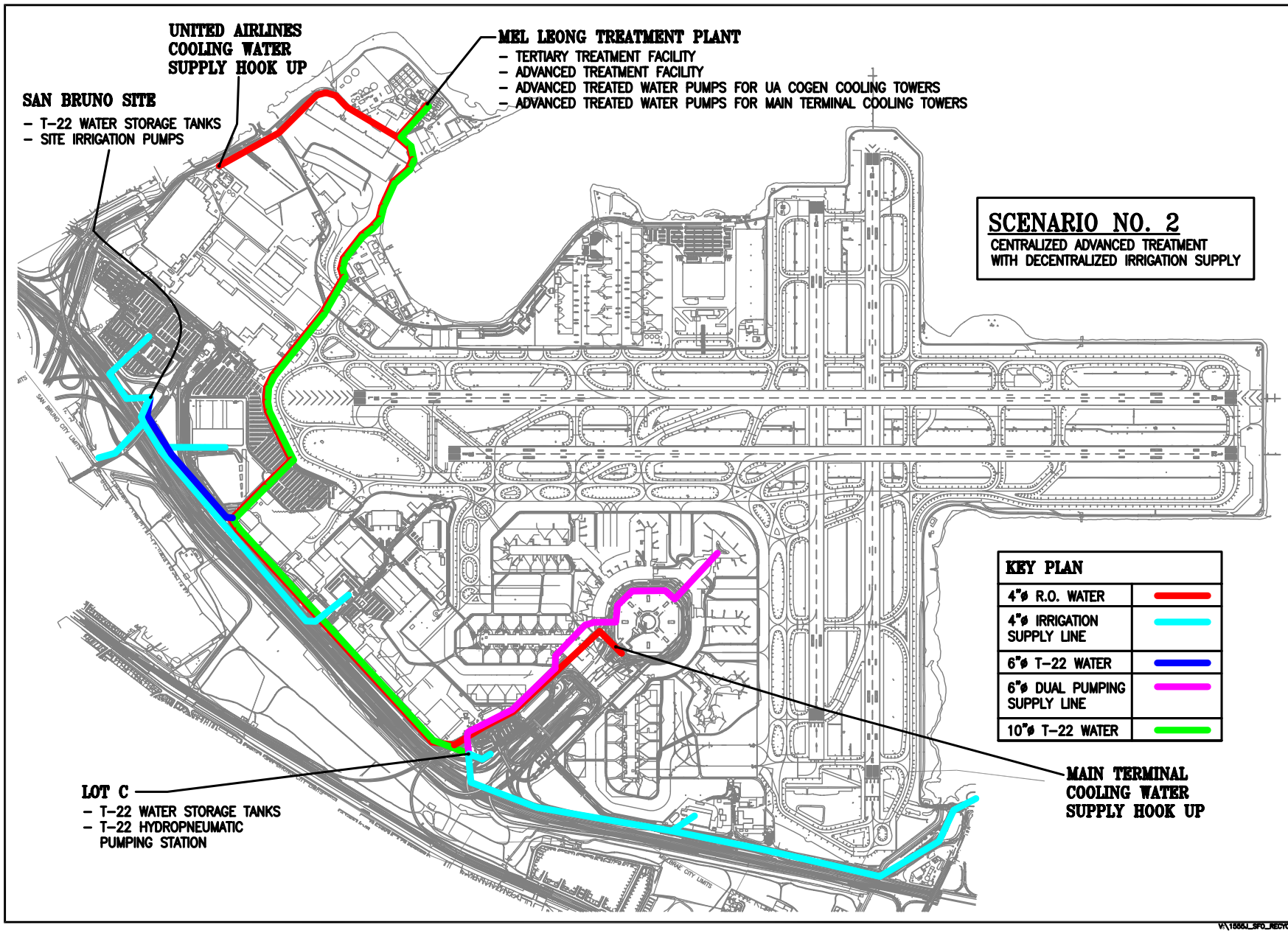

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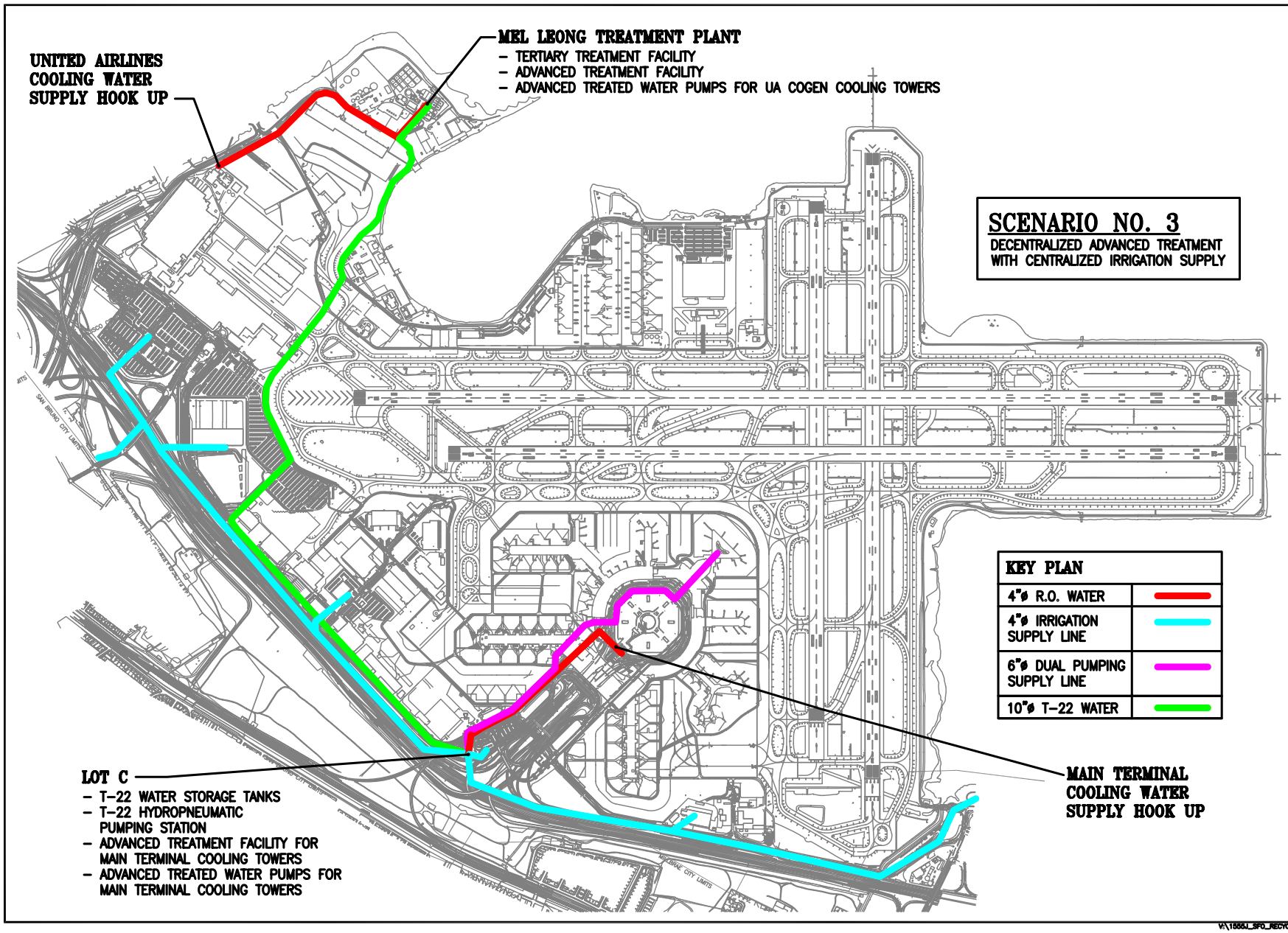

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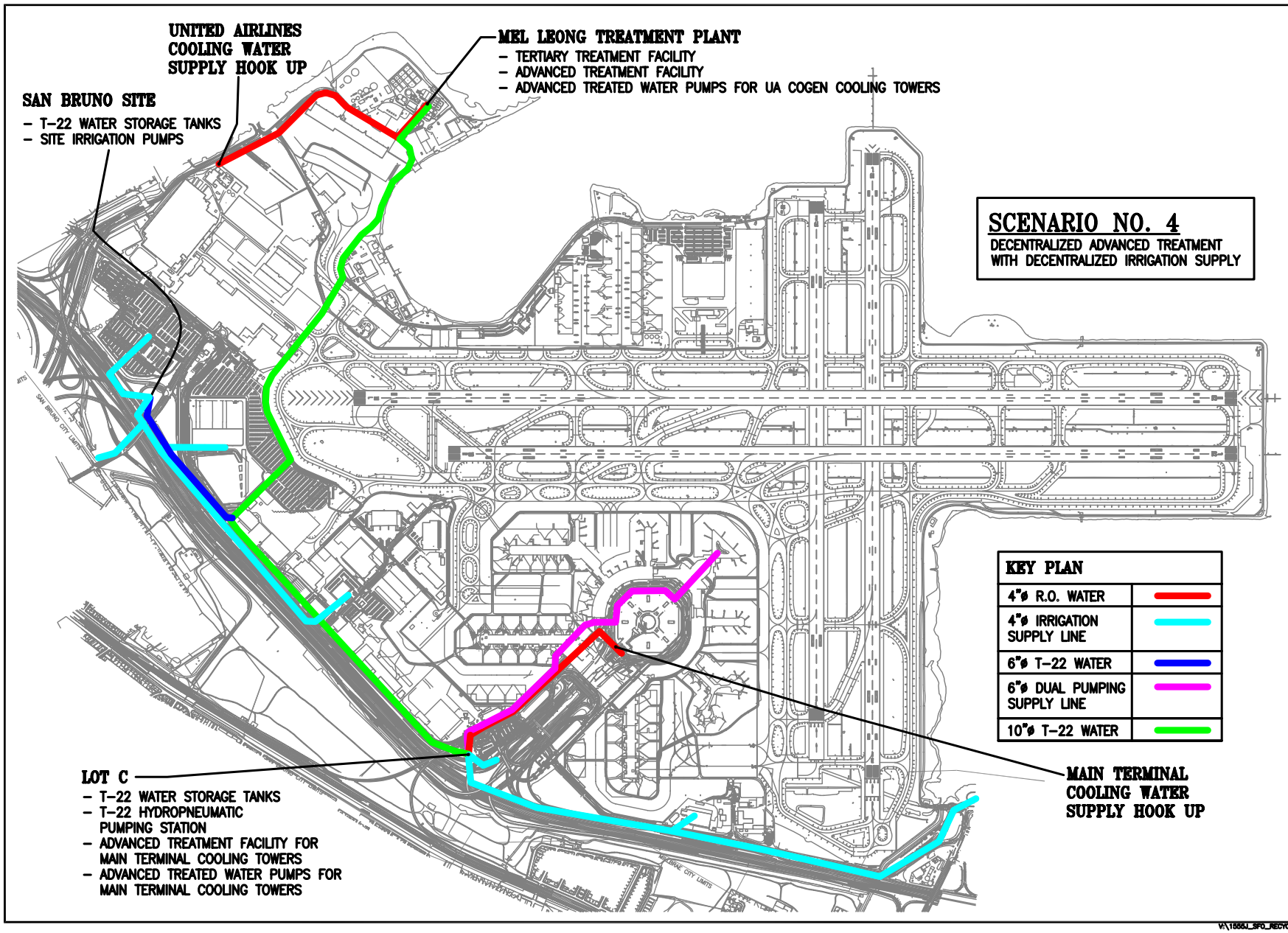

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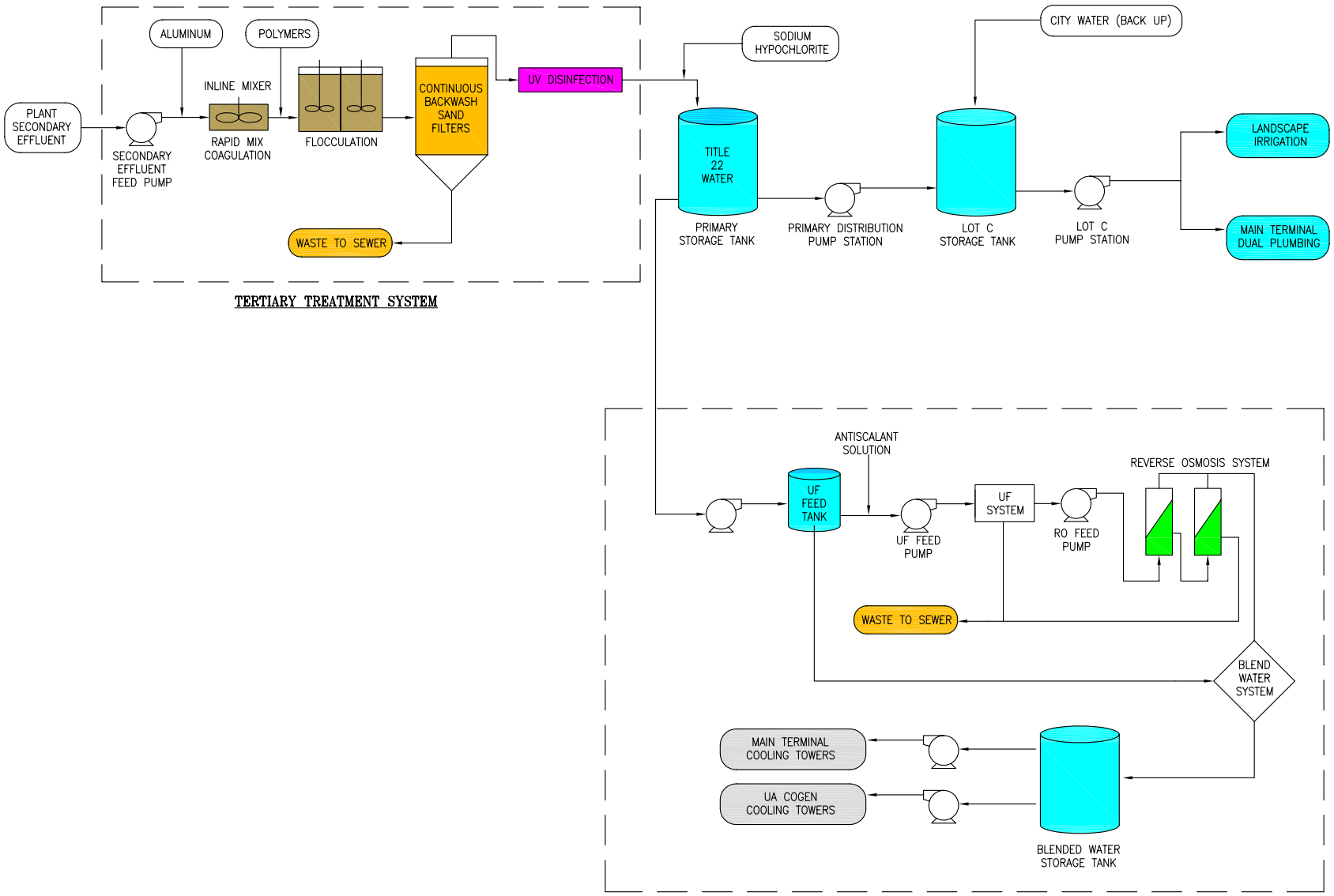

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SHEET TITLE: SCHEMATIC PROCESS FLOW DIAGRAM SCN. 1 FIGURE 7.5

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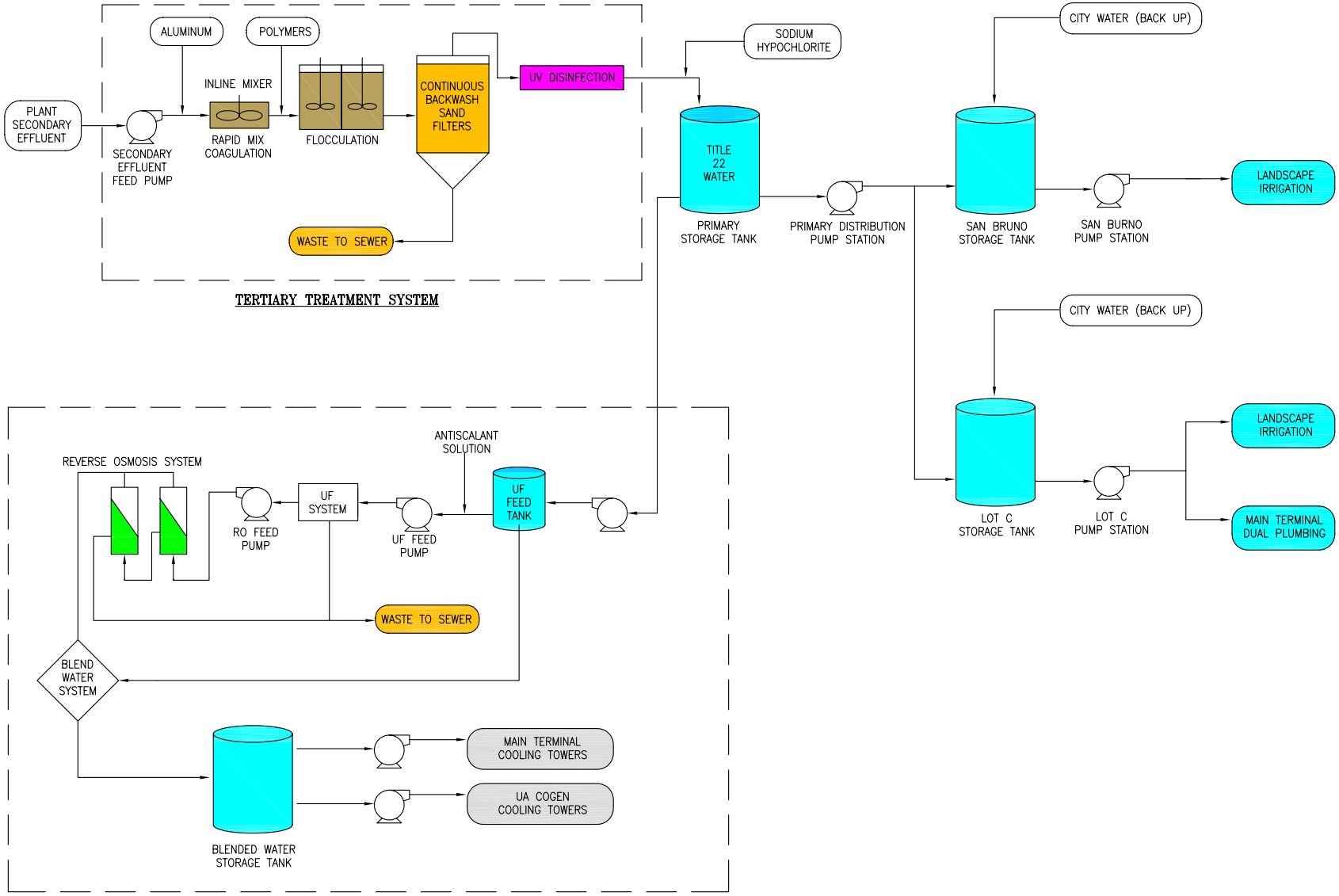

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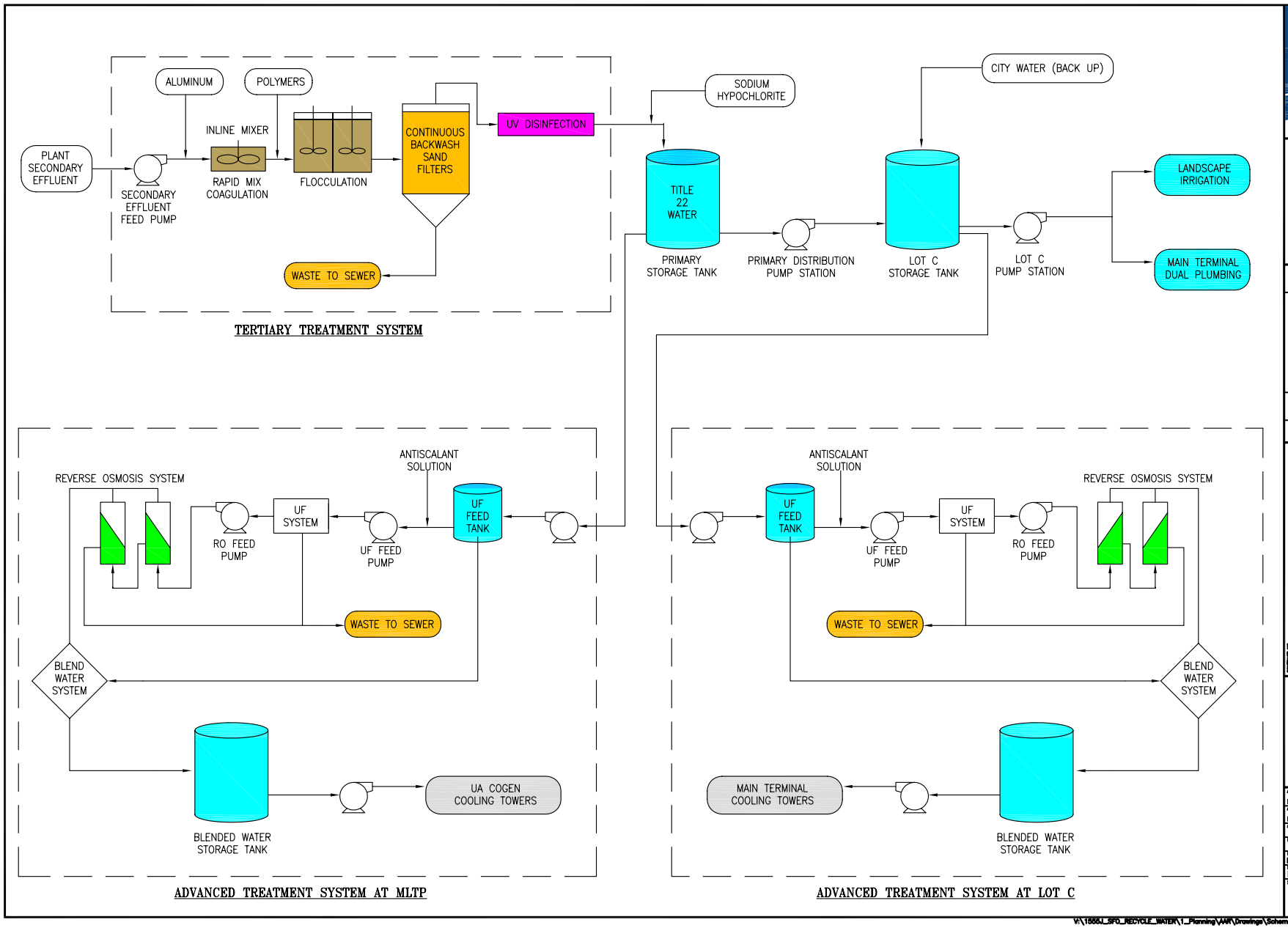

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SHEET TITLE: SCHEMATIC PROCESS FLOW DIAGRAM SCN. 3 FIGURE 7.7

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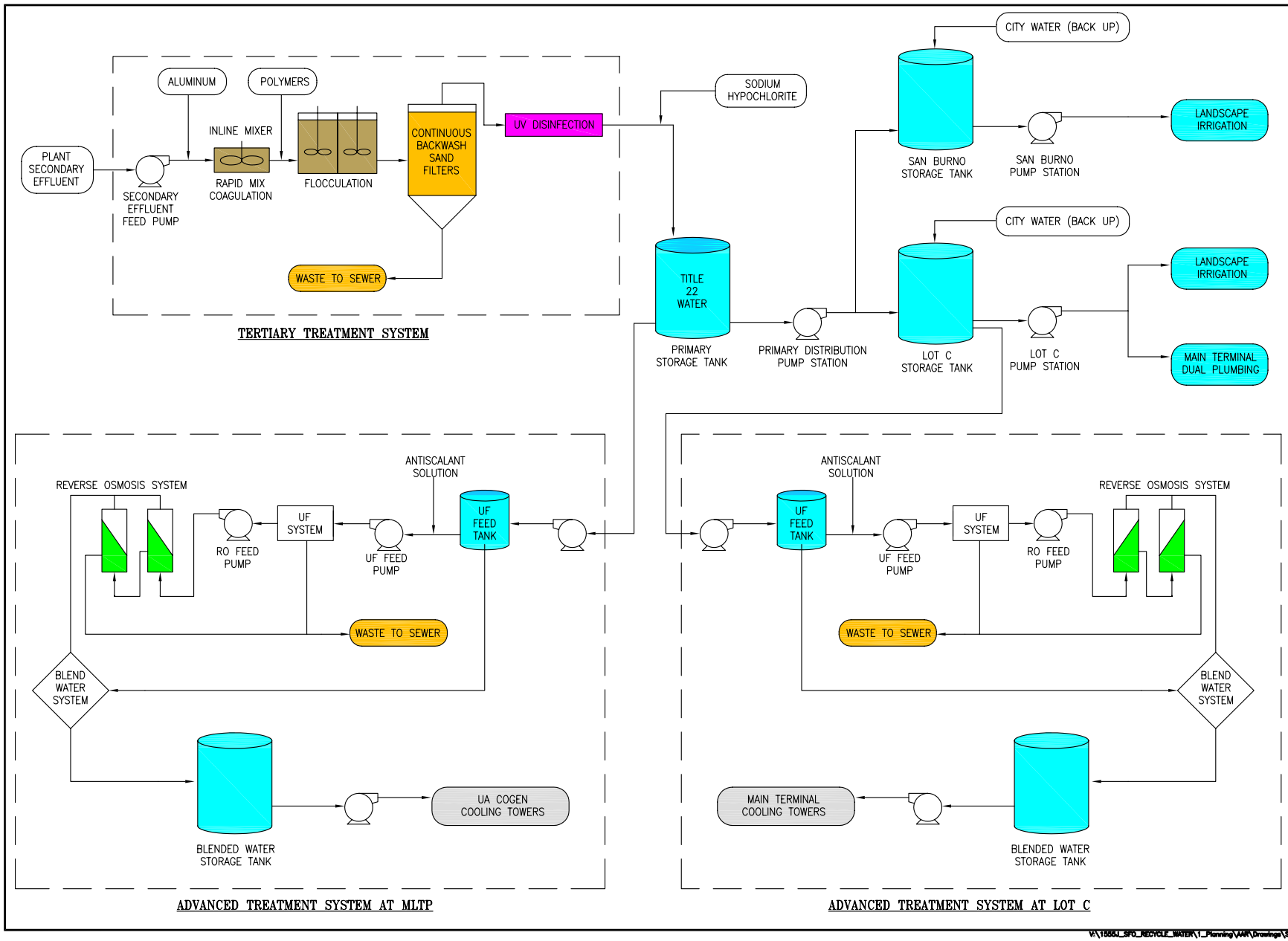

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8.0 Recommendation

8.1 Introduction

This section presents a benefit-to-cost ratio analysis for each scenario, as described in the previous section, and recommends an alternative.

8.2 Benefit-to-Cost Ratio Analysis

Table 8-1 presents a summary of the benefit-to-cost ratio analysis for the four scenarios.

Table 8-1
Benefit-to-Cost Ratios of Four Combined Scenarios

Item	Criteria Weights	Scenario No.1	Scenario No.2	Scenario No.3	Scenario No.4
Raw Benefits Scores					
Implementability	2	2	2	3	2
Reliability	4	3	3	3	4
Ease of Operation and Maintenance	5	4	3	3	3
Impacts to cultural and Environmental Resources	1	2	2	2	2
Geologic and Contamination Hazards	2	2	2	2	2
Staff Training Requirements	3	3	3	3	3
Implementation Schedule	3	2	2	3	3
Weighted Benefit Scores		57	52	55	59
Net Annualized Cost (millions)		$1.8	$1.9	$1.8	$1.9
Benefit-to-Cost Ratio		31.7	27.4	30.6	31.1

(See section 3.4: “Evaluation criteria for measuring benefits” for the value of each score)

Since the net annual costs of each scenario are within a 4% range, other factors had to be evaluated to determine the best alternative. Scenarios 3 and 4 (decentralized treatment) offer more flexibility in phasing and implementation of the project. A tertiary treatment facility and distribution system can be constructed, independent of advanced treatment, to provide Title 22 water to the end user. Scenarios 1 and 2 (centralized treatment), on the other hand, would require tertiary and advanced

distribution systems to be installed simultaneously. In the event that an advanced treatment facility is not constructed, all additional piping costs for the advanced treatment distribution system would be wasted.

A decentralized irrigation supply (scenarios 2 and 3) is desirable because of ease of control and ability to accommodate future increases in irrigation demands. A centralized irrigation supply (scenario 1 and 4) would require a large pump station capable of delivering water to far reaching locations at working pressures of 60-70 psi. More supply points and smaller pumps would make the irrigation supply system more robust and manageable.

8.3 Recommended Alternative

Given the analysis, scenario 4 is the recommended alternative. A proposed system configuration, process flow diagram, and layout are shown in Figures 8.1 through 8.4. (All figures in this section are provided at the end of this section.)

8.3.1 Preliminary Design Criteria

Lists of major equipment and preliminary design criteria are presented below. These parameters are based on planning-level analyses. Further refinement of process elements and detailed design criteria will be established during the CER report phase of the project.

8.3.1.1 Tertiary Treatment Facilities

Based on the recommended alternative, a CER will be prepared for the tertiary treatment processes that include rapid mix coagulation, upflow deep-bed continuous backwash filtration, UV disinfection, and residual disinfection with sodium hypochlorite. The backwash wastewater from the continuous backwash filters would be recycled to the equalization tanks at MLTP.

Table 8-2 presents a summary of the preliminary process design criteria for producing "tertiary disinfected recycled water." The design treatment capacity of 0.75 MGD is adequate for ADMM demands of the landscape irrigation, dual plumbing and cooling towers.

Table 8-2
Preliminary Design Criteria for Tertiary Treatment Facility

Criteria	Value
Tertiary Treatment Feedwater Pump Station	
Firm Capacity, gpm (mgd)	525 gpm (0.75 mgd)
Number of Pumps, duty (standby)	2 (1)
Approximate Motor Size, hp	7.5
Pretreatment	
Coagulant	Alum
Average Coagulant Dose, mg/L	30.0 (25 to 40)
Rapid-mix Coagulation	
Type	Inline Mechanical Mixer

Number, duty (standby)	1 (1)
Mixing Time, seconds	2
Headloss, feet of water	1 to 3
Pipe Velocity, ft/second	2 to 6
Flocculation	
Type	Vertical Impeller Mixer
Detention Time, minutes	20
Number of Stages per train	4
Approximate Flocculator Power, hp each	3
Filtration	
Type	Continuous Backwash, Upflow Sand Filters
Bed Depth, Inches	60
Maximum Hydraulic Loading Rate, gpm/ft²	4
Air compressors for Airlift Pump	
Number, duty (standby)	1 (1)
Approximate Motor Size, hp	20
Disinfection	
Type	Low-pressure/High-Intensity UV Irradiation
Number of Tube	40
Number of Modules, duty (standby)	1 (1)
Minimum UV bioassay dose, mJ/cm²	80
Minimum Ultraviolet Transmittance @ 253.7 nm	65%
Chlorination for Distribution System	
Type	Sodium Hypochlorite Solution
Dose, mg/L average (range)	3 (1 to 3)

8.3.1.2 Advanced Treatment Facilities

Preliminary design criteria for advanced treatment facilities are presented in Table 8-3. The recommended advanced treatment system for producing recycled water for the cooling towers include pressurized MF/UF, chemical conditioning, RO and residual disinfection with sodium hypochlorite. The rejects from MF/UF and RO systems would be discharged to the sewer and returned to the MLTP for treatment. The effect of the rejects on the treatment plant performance is not expected to be significant because of the relatively small capacity compared to the total flow. It is recommended that the design criteria be refined through detailed analyses prior to completing the design.

Table 8-3
Preliminary Design Criteria for Advanced Treatment Facility

Criteria	Value
Feedwater Pump Station	
Capacity, gpm (mgd)	210 gpm (0.3 mgd)
Number of Pumps, duty (standby)	1 (1)
Microfiltration/Ultrafiltration	
Type	Pressurized, Low Pressure

Membrane Type	Hollow Fiber, PVDF
Recovery	88%
Flux Rate, gfd	36
Approximate Backpulse Interval, minutes	25
Approximate Clean-In-Place Interval, days	30
95th Percentile Filtrate Turbidity, NTU	<0.1
Reverse Osmosis	
Membrane Material	Polyamide Thin-film Composite
Number of stages	2
Membrane Element Size, diameter x length	8” x 40”
Operating Pressure, psig	175
Recovery	77.5%
Flux, gdf	10 to 12
Approximate Clean-In-Place Interval, days	180

8.3.2 Pump Stations

Preliminary sizes and locations of pump stations were established in the analysis of alternatives for the distribution system. The preliminary results are listed in Table 8-4.

Table 8-4
Estimated Requirements for Pump Stations

Pump Name	Estimated Flow (gpm) and Pressure (psig) at Peak Demand
Secondary Effluent Pump Station	525 @ 35
Primary Pump Stations	425 @ 50
San Bruno Pump Station	300 @ 85
Lot C Pump Station	450 @ 95
UAL CT Pump Station	120 @ 95
Terminal CT Pump Station	100 @ 95

8.3.3 Storage Tanks

Storage sizing will be performed during preliminary design phase. For planning cost estimating purposes, the main Title 22 water storage tank at MLTP is 300,000 gallons, the advanced treated water storage tank at MLTP is 100,000 gallons. The San Bruno site storage tank is 75,000 gallons. Three of the existing underground storage tanks will be retrofitted to store the Title 22 water and the remaining two will store the advanced treated water.

8.3.4 Operating Strategy

The recycled water treatment systems would be designed for automatic control as much as possible to minimize labor requirements. A PLC would be provided for control and monitoring of the treatment systems. The recycled water treatment system PLCs would be located in a new or existing electrical and control building at respective sites. Feasibility of completely integrating monitoring and control functions of RWF to the existing MLTP SCADA system will be explored to reduce the number of new operators required to operate the new facility. Integrating RWF controls to existing SCADA will also allow monitoring and control of RWF from the existing main control room and make it possible to remotely initiate operation of the treatment systems, including changing control set points, performing remote manual controls and viewing all treatment plant alarm data, discrete and analog variables statuses.

Local control panels would also be provided for automatic and local manual control. The recycled water treatment system PLCs would monitor status and alarm signals from the local control panels. In general, emergency shutdown of equipment due to alarm conditions (low discharge flow, high discharge pressure, motor overload, pump seal water failure, etc.) would be hardwired and would occur at the associated motor control center (MCC) or local control panel. These alarms conditions would require manual reset at the MCC or local control panel. Manual reset of these alarms at the MCC or local control panel would automatically reset the alarm at the PLC. When the PLC receives these alarm signals for equipment it is allowed to control, the PLC would remove the run command and would reset the equipment control mode to manual. Alarms that do not result in equipment protection shutdown would reset automatically when the alarm conditions no longer exist.

8.3.4.1 Tertiary Treatment Facilities

The tertiary treatment facilities would be operated to match seasonal demands. During peak conditions, up to 0.75 mgd would be consumed for irrigation, dual plumbing, cooling towers and MLTP needs. The peak demands are expected to occur during summer months when irrigation and cooling towers demands peak. Portions of the treatment facilities would be idled when demand is reduced. However, recycled water production would remain relatively constant throughout the day. Storage tanks in the distribution system would be used to equalize daily demands.

The primary pump station located at the treatment facility would operate as needed to meet demands and fill storage tanks in the distribution system. Other pump stations in the system would operate as needed to maintain pressure in the system.

It would be necessary to flush tanks and pipes periodically to maintain water quality. The water flushed from the system could be discharged into the sewer system and retreated at the MLTP. Blowoff valves would be provided at low points and dead ends of the distribution system to facilitate flushing.

8.3.4.2 Advanced Treatment Facilities

The advanced treatment system would operate daily to provide advanced treated water supply to the cooling towers. Backwash and brine produced in the plant would be discharged to the sewer system and retreated at the MLTP.

8.3.5 Power Supply

The primary power supply for the treatment and distribution systems would be supplied off the SFIA electrical power grid. The treatment facility and each pump station would require a separate power feed that includes transformers, meters, and switchgear.

Standby power for the treatment facility and primary pump station can be provided using diesel engine generators. At remote pump stations, receptacles would be provided so portable generators could be connected.

8.3.6 Corrosion Control

Measures to control corrosion would be required to prevent premature degradation of the recycled water system. Corrosion may result from contact between incompatible materials, contact with corrosive soils, salt spray from the ocean, chemicals used in treatment, and the water itself. Each of these potential sources of corrosion should be addressed in the design of the facilities.

8.3.6.1 Distribution Pipes

Buried pipelines shall be protected using corrosion resistant material such as PVC. For construction of ductile iron pipe, corrosion measures can include cement mortar lining, asphalt varnish coating, polyethylene encasement, and/or cathodic protection.

8.3.6.2 Pump Stations

Buried pipelines at pump stations should be protected in the same manner that distribution pipelines are protected.

Aboveground components of pump stations should be protected using appropriate coatings and shelters. In addition, all dissimilar materials shall be separated to prevent galvanic corrosion.

8.3.6.3 Storage Tanks

Protective coatings should be applied to storage tanks to prevent corrosion.

8.3.6.4 Tertiary Treatment Facility

Corrosion protection measures at the tertiary treatment facilities would be similar to measured used at the pump stations and storage tanks. In addition, the system should be design to prevent corrosion from chemicals used for treatment.

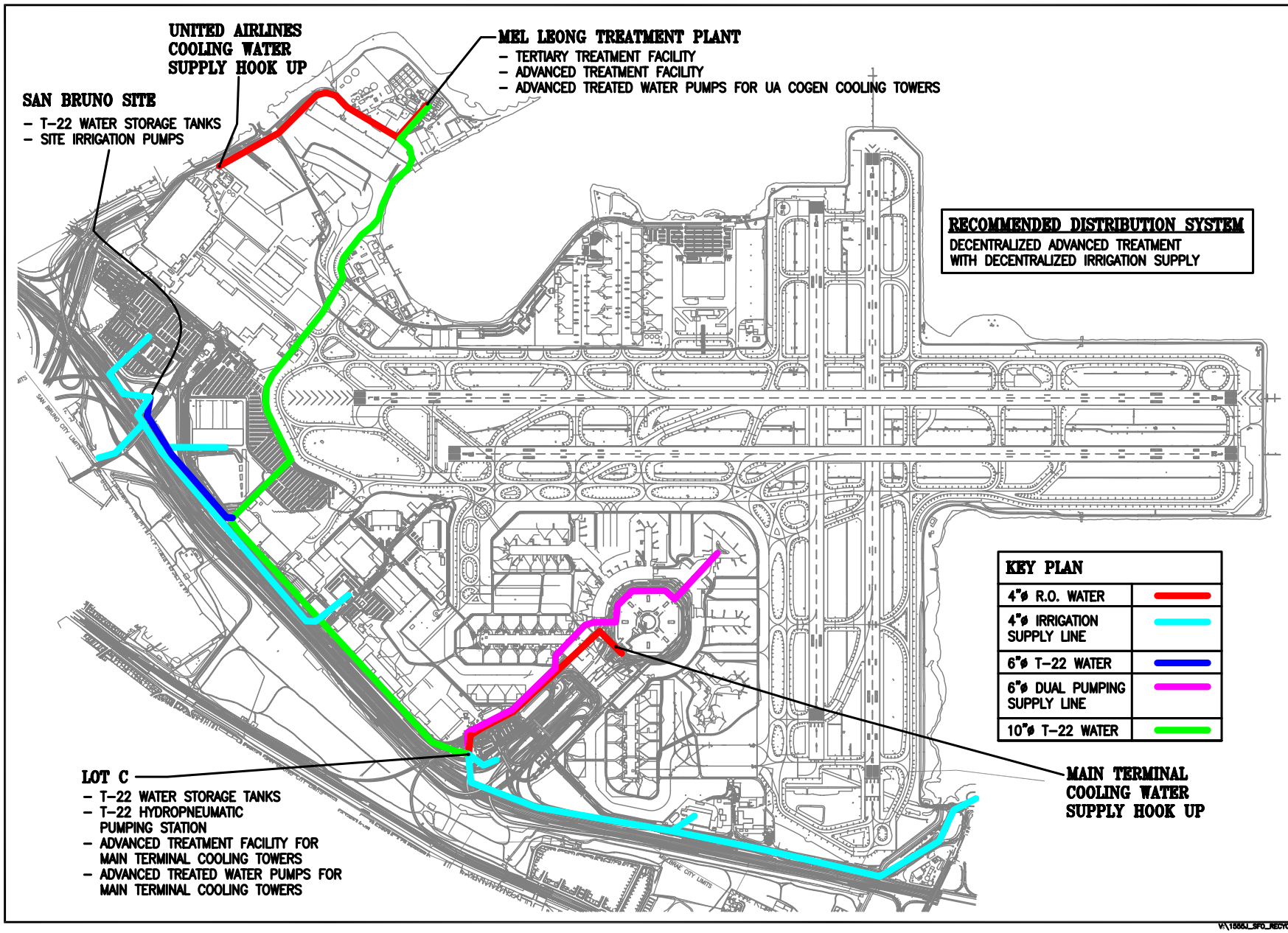
If the facilities must be constructed underground, special protection and waterproofing measures may be required for concrete structures, especially if they are constructed in highly corrosive soils. Likewise, materials of construction should be selected carefully if the facility will be exposed to salt spray from the ocean.

8.4 Construction Phasing

Construction of the recommended alternative for Recycled Water Project can be phased-in to construct and commission the tertiary treatment facility and distribution system to Lot C for dual plumbing supply first and the irrigation supply and advanced treatment to follow. Table 8-5 shows the estimated cost for the first phase of construction for dual plumbing.

Table 8-5
Estimated Construction Costs for Dual Plumbing Phase

Item	Scenario 4
Tertiary Treatment Facilities	
Pretreatment	$100,000
Continuous Backwash Sand Filtration	$525,000
UV Disinfection	$415,000
Chemical System	$ 75,000
Structural Foundation and Piles	$380,000
Support Facilities	
Secondary Effluent Pump Station	$71,000
Secondary Effluent Force Main	$22,500
Modification of (E) Effluent Pump Station	$25,000
Title 22 Distribution System	
Pump Stations	
Primary Pump Station	$135,000
Lot C Pump Station w/Hydro Tanks	$250,000
Storage Tanks	
Main Storage Tank	$700,000
Lot C Storage Tank Retrofit	$500,000
Electrical and I&C Work	$375,000
Mechanical and Process Piping	$250,000
Structural and Site Work	$100,000
Subtotal	$3,923,500
Mob/Demob/Bonds/Insurance	$ 150,000
Allowances	$ 50,000
Subtotal	$4,123,500
15% Contractor Markup	$ 618,500
15% Contingency	$ 618,500
Subtotal	$5,360,500
Pipeline Work	$2,353,000
Total Construction Cost	**$7,713,500**



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SHEET TITLE: RECOMMENDED DISTRIBUTION SYSTEM FIGURE 8.1

APPROVED: DEPUTY AIRPORT DIRECTOR

APPROVED: PRINCIPAL

DESIGNED BY: CHECKED BY:

DRAWN BY: EL DATE: 3/31/2009

SCALE: 1"=600' REVISION: D

SHEET NUMBER SEQUENCE NUMBER ___ of TBD

V:\1555_SFO_RECYCLE_WATER\1_Planning\AAR\Drawings\Figure 8.1.dwg

R_TYPE_OF_ISSUE

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Note:
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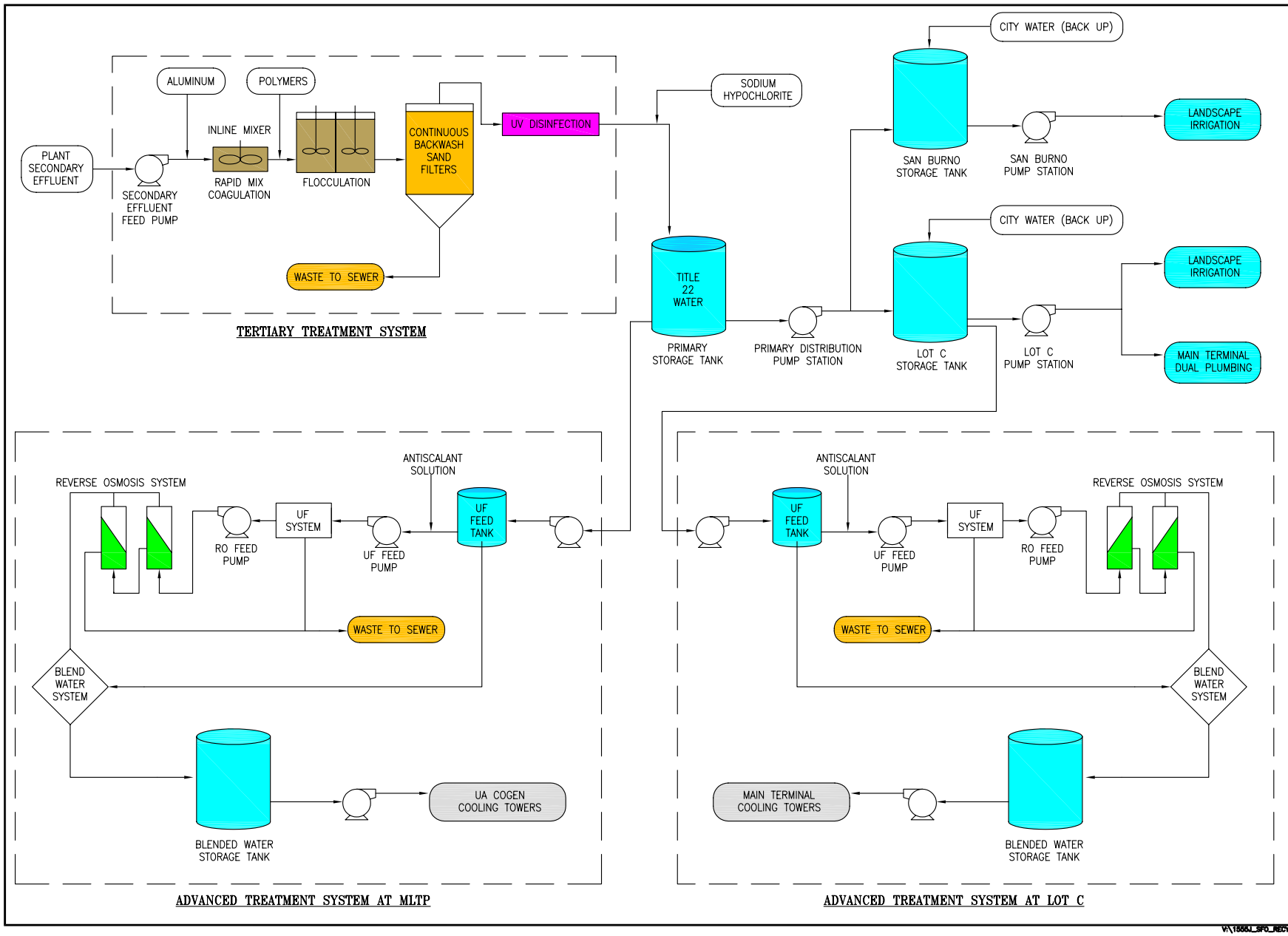

SFO
San Francisco International Airport

Key Map

REVISIONS

NO.	DATE	DESCRIPTION	BY

CONTRACT NO. 8541

PROJECT TITLE: AIRPORT RECLAIMED WATER – ALTERNATIVE ANALYSIS REPORT

SHEET TITLE: RECOMMENDED PROCESS FLOW DIAGRAM FIGURE 8.2

APPROVED: DEPUTY AIRPORT DIRECTOR
APPROVED: PRINCIPAL
APPROVED:

DESIGNED BY: EL
DRAWN BY: EL
CHECKED BY:
DATE: 3/25/2009
SCALE: NONE
REVISION: D
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SITE MAP

TERTIARY TREATMENT FACILITY IN MLTP

SFO

San Francisco International Airport

Key Map

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REVISIONS

CONTRACT NO. 8541

PROJECT TITLE: AIRPORT RECLAIMED WATER – ALTERNATIVE ANALYSIS REPORT

SHEET TITLE: RECOMMENDED SITE FIGURE 8.3

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DRAWN BY: EL | DATE: 3/25/2009

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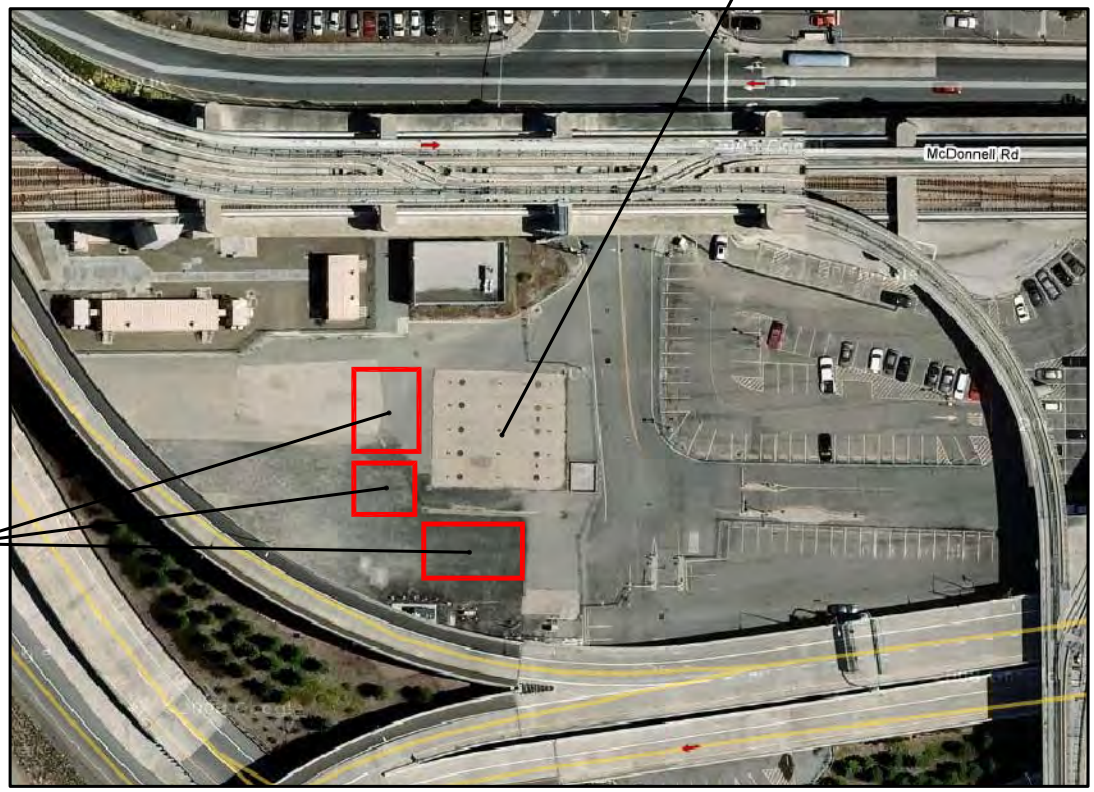
SITE MAP



LOT C

SFO
San Francisco International Airport

Key Map

NO.	DATE	DESCRIPTION	BY

REVISIONS

CONTRACT NO. 8541

PROJECT TITLE: AIRPORT RECLAIMED WATER – ALTERNATIVE ANALYSIS REPORT

SHEET TITLE: RECOMMENDED SITE FIGURE 8.4

APPROVED:
DEPUTY AIRPORT DIRECTOR
APPROVED:
PRINCIPAL
APPROVED:

DESIGNED BY: | CHECKED BY:
DRAWN BY: EL | DATE: 3/25/2009
SCALE: NONE | REV. D | REVISION
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V:\1555_SFO_RECYCLE_WATER\1_Planning\AAR\Drawings\Figure 8.4.dwg

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9.0 Project Implementation

9.1 Introduction

This section describes some of the implementation support tasks, including the project implementation plan and schedule, phasing, recommendations for additional studies, conceptual engineering report outline, and cross-connection control evaluation study plan.

9.2 Project Implementation Plan and Schedule

The preparation of the AAR is the first step in the implementation of the SFIA Recycled Water Project. The project activities following the AAR will include conducting additional studies as required, preparation of the CER, environmental documentation and permitting, funding, financing, detailed design, contract bidding, construction, and start of operation. A preliminary schedule of major implementation activities or tasks is presented in Table 9-1.

Table 9-1
Implementation Activities and Schedule Summary

Tasks	Schedule
Program Management	Ongoing
Environmental Documentation and Permitting	2009
Engineering and Construction	2009
Alternative Analysis	2009 (Completed)
Additional Studies	2009 (As required)
Conceptual Engineering	2009
Detail Design/Bidding	2009-2010
(Tertiary Treatment Design Phase)	2009
(Advanced Treatment Design Phase)	2010
Construction Permitting	2010
Construction	2009-2010
(Tertiary Treatment Construction Phase)	2010
(Advanced Treatment Construction Phase)	2011
Commissioning	2011-2012
(Tertiary Treatment)	2011
(Advanced Treatment)	2012
Start of Operation	2011-2012
(Tertiary Treatment)	2011
(Advanced Treatment)	2012

9.3 Project Phasing

The primary objective of this project is to deliver recycled water to Terminal 2 by February 2011. Terminal 2 is currently being renovated and all water closets and urinals will be plumbed for recycled water. Given that, the project will be phased into three separate contracts: main distribution, treatment facilities, and irrigation/cooling towers distribution.

The main distribution contract will construct the pipe from the MLTP to Lot C, and the delivery pipe from Lot C to Terminal 2. It will also install distribution pump stations, hydro-pneumatic pumps, and storage tanks, as well as retrofit the existing 5 -40,000 gallon underground storage tanks. This project will be completed by December 2010.

The treatment facilities contract will construction the tertiary treatment facility at the MLTP. This project, likewise, will be completed by January 2011.

The third phase will construct all the irrigation and cooling tower supply pipes. Given the priority of the Terminal 2 Renovation project, this contract will be completed by December 2011.

9.4 Recommendations for Additional Studies

During the course of the alternative analysis process several additional required studies were identified. These studies include:

- Additional sampling of MLTP secondary effluent for pertinent parameters
- Monitoring of water demands for landscape irrigation
- Piloting of sand filtration system using a DynaSand D2 mobile pilot unit

9.4.1 Additional Sampling of Secondary Effluent

The historical secondary effluent data does not provide adequate information for design of the RO system. Periodical sampling should be conducted for general minerals, TDS, silica and other constituents required in Title 22 for cooling tower usage.

9.4.2 Monitoring of Water Demands for Landscape Irrigation

The landscape irrigation demands should to be monitored and recorded in order to confirm the demand and have accurate flow requirement during design. BOE can assist in provisioning flow measuring and recording instrumentation to achieve this objectives.

9.5 Conceptual Engineering Report Outline

The CER will be prepared to develop the preferred alternative identified in the AAR. The CER serves as the basis of design for the recommended project. A sample outline (table of contents) of a CER from a similar recycled water project is shown in Table 9-2.

Table 9-2
Example Table of Content

Section	Title
1	Introduction
2	Regulatory Requirements
3	Interfacing with Existing Facilities
4	Process Description and Design Criteria
5	Process Control, Instrumentation and Monitoring
6	Civil Design Criteria
7	Structural and Geotechnical Design Criteria
8	Mechanical Design Criteria
9	Electrical Design Criteria
10	Construction Aspects and Preliminary Cost Estimates
11	Recommendations for Final design
12	Operations Plan and Staffing Requirements
	Appendices

9.6 Cross-connection Control Evaluation Study Plan

A cross-connection is an unapproved interconnection between a potable water line and a piped system containing some non-potable fluid. A cross-connection could allow the non-potable fluid to flow under pressure or be siphoned into the potable water system. The use of recycled water in the proximity of potable water creates the risk of cross-connection between the water systems and the contamination of potable water.

State laws and regulations help to protect public domestic water systems and prevent cross-connections. Adopted by the Department of Health Services ("DHS"), Title 17 of the California Code of Regulations ("CCR") specifies the installation of backflow prevention devices or features, depending on the conditions on the use site and the degree of public health risk. Such approved devices (listed in an increasing level of protection) include: double-check valve assembly, reduced pressure principle backflow prevention device, and an air-gap. The water user may choose a higher level of protection than required by the water supplier.

Title 22 of CCR has specific requirements for dual-plumbed recycle water systems to prevent the unintentional misuse of recycled water and the cross-connection of recycled water and potable water within buildings and landscaping. The dual-plumbed section uses a combination of postings, plumbing access restrictions, plumbing labels, supervision, periodic inspection, and testing to minimize the chance of misuse or cross-connection.

The California Plumbing Code (Appendix J) has cross-connection requirements specifically for the installation, construction, alteration, and repair of recycled water systems intended to supply toilets (water closets), urinals, and trap primers for floor drains and floor sinks. The recycled water system shall not have any connections to the

potable water system. Before the building may be occupied, the installer shall perform an initial cross-connection test using a temporary connection to a potable water source and the test shall be ruled successful before the recycled water supply can be connected.

In addition to the cross-connection requirements at the user premises, there are guidelines from DHS, United State Environmental Protection Agency ("USEPA"), California-Nevada ("CA-NV") Section America Water Works Association ("AWWA") that contain recommendations regarding pressure, minimum burial depth, separation distances from other utilities (especially potable water), pipe identification, isolation valves, air relief and vacuum valves, and blow-off assemblies. The CA-NV Section AWWA also has guidelines for the on-site retrofit of facilities using disinfected tertiary recycled water for the planning and designing retrofits of facilities to use recycled water.

During the pre-design stage of the RWP, a cross-connection control study should be conducted to evaluate each user site to ensure that all the above mentioned requirements and guidelines are complied with, and that the design of the distribution system and retrofits of facilities do not allow any potential for cross-connection.

SF PUC Water Supply and Treatment Division's Water Quality Bureau ("WQB") has jurisdiction over cross-connection when an entity using their water intends to produce and distribute recycled water. SFIA will need to make arrangement and contact WQB to initiate the permitting and approval of the cross-connection systems during the CER phase of the project.

San Francisco Airport Tertiary Treatment Plant Life Cycle Analysis

CEE 226: Life Cycle Assessment for Complex Systems

December 6, 2012

Amy Allen, Jennifer De Bruijn, Ariel Hippe, Sijia Tao, Juliana Vélez

Outline

- Overview of Project
  - Life Cycle Assessment Project Goals
    - Water Comparison
      - Energy Comparison
        - Emissions Comparison
          - Cost Comparison
    - Conclusions

SFO Mel Leong Treatment Plant

Industrial Wastewater Treatment Plant (IWTP)

- Treats wastewater from industrial sources at the Airport
- Treats first flush of storm water runoff
- Average flow is 1.15 mgd

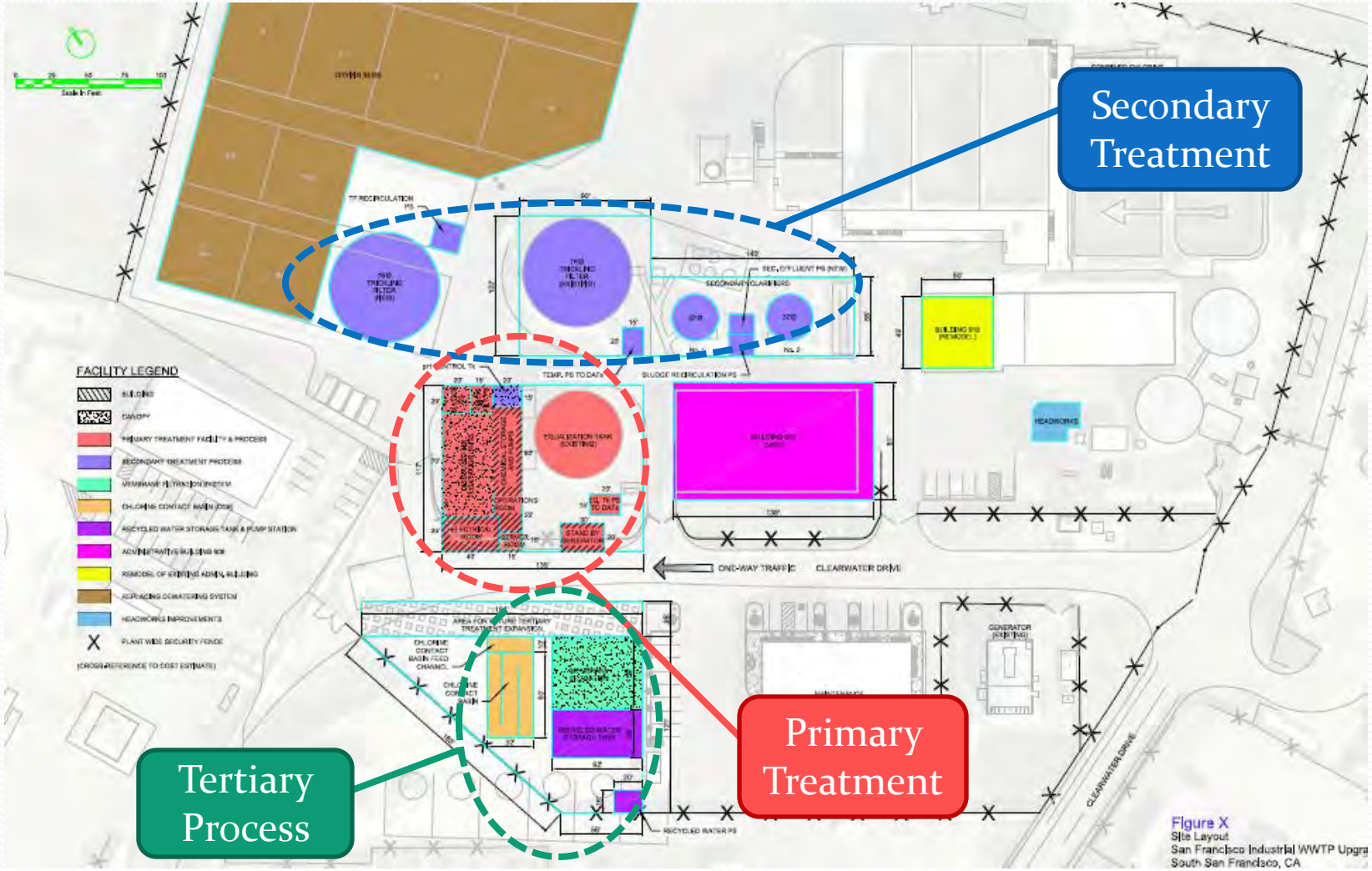
Sanitary Sewer Wastewater Treatment Plant

- Secondary treatment for wastewater from terminal buildings, aircrafts, restaurants, shops, and other Airport facilities
- Design treatment capacity of 2.2 mgd

Proposed New System

- New Industrial Wastewater Treatment Plant
- New Tertiary Treatment Plant
  - Filtration Membrane
  - Treat the waste water into re-useable
  - Water used for domestic use in toilets/urinals, landscaping, cooling towers, and potentially ground water recharge

Proposed New System



Project Goals

- Life cycle assessment of the new tertiary water treatment plant proposed by SFO.
- Determine the better option:

  Tertiary treatment and reuse of water

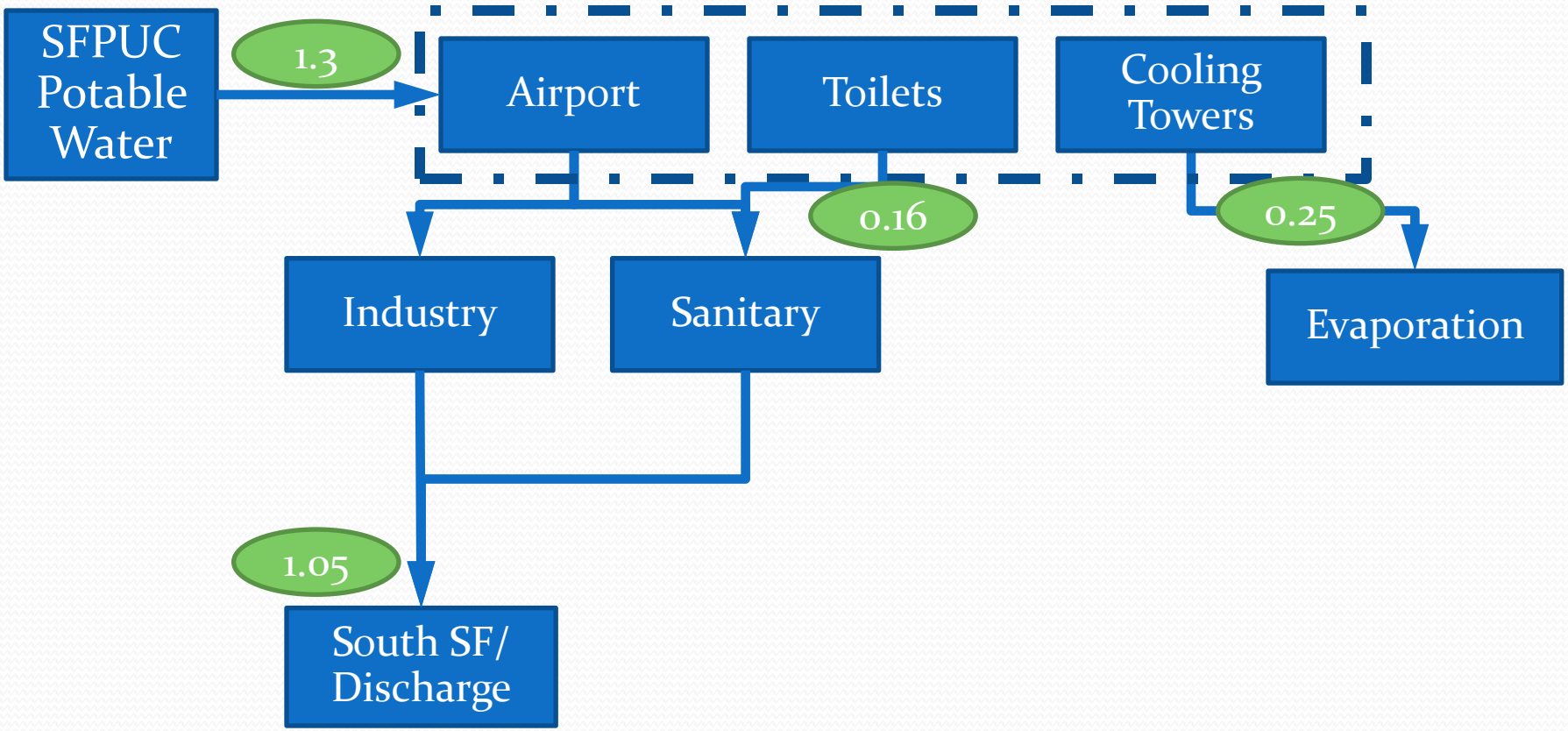
  vs

  Purchase of water from SF Public Utilities Commission

  - **Water Use**
  - **Energy**
  - **Emissions**
  - **Costs**
- Functional Unit – water consumption of SFO – 1 mgd

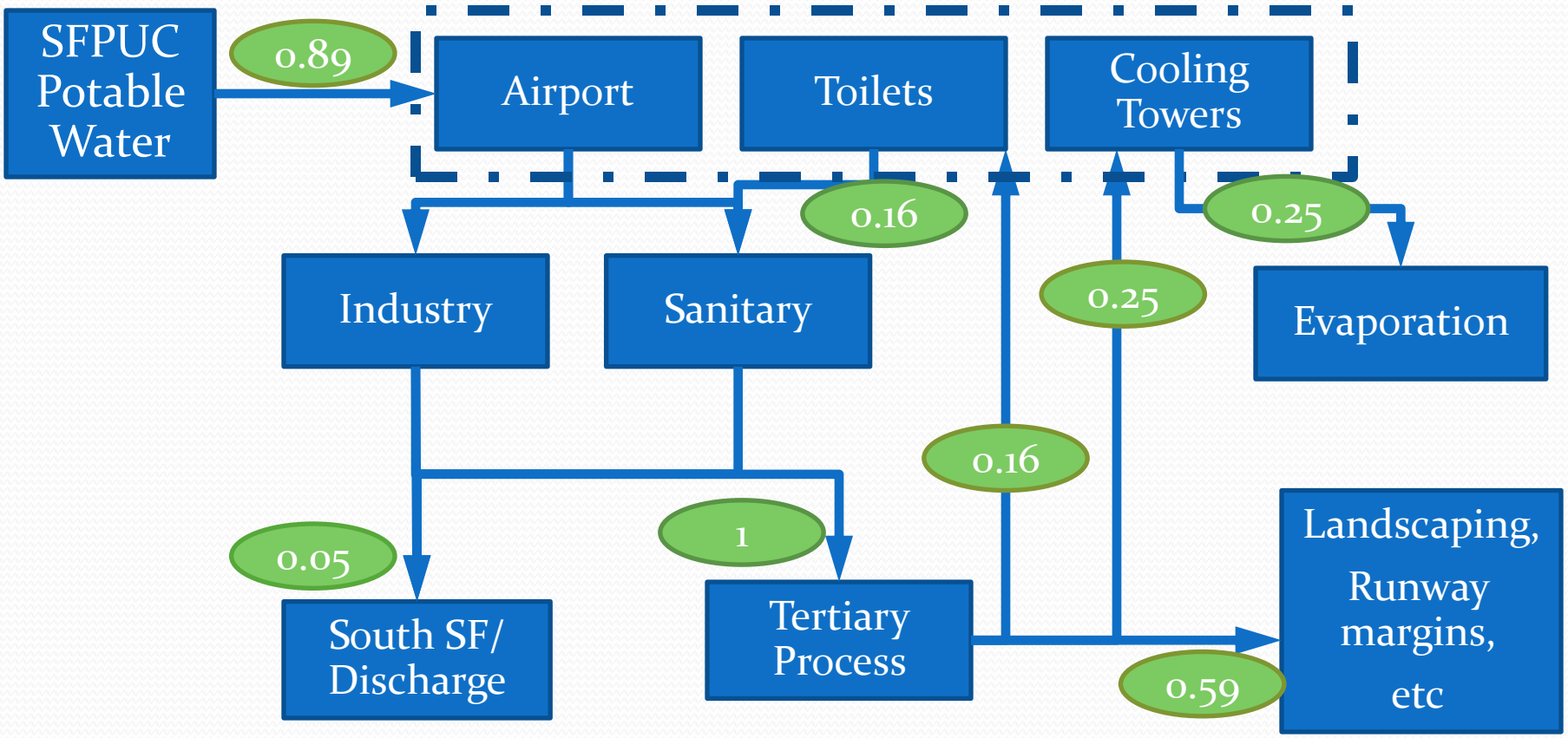
Process Flow Diagram-Current

All numbers in million gallons per day



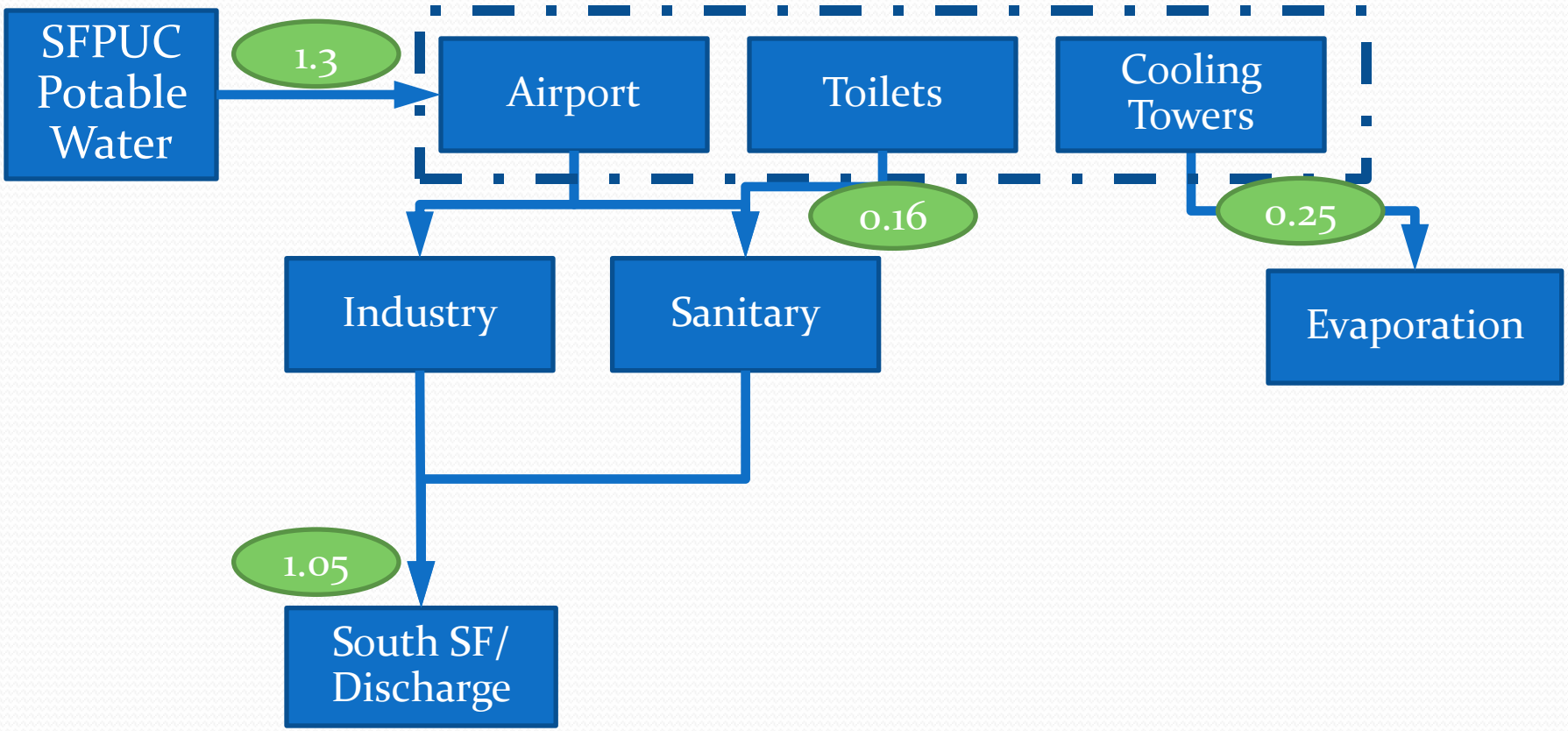
Process Flow Diagram-New

All numbers in million gallons per day



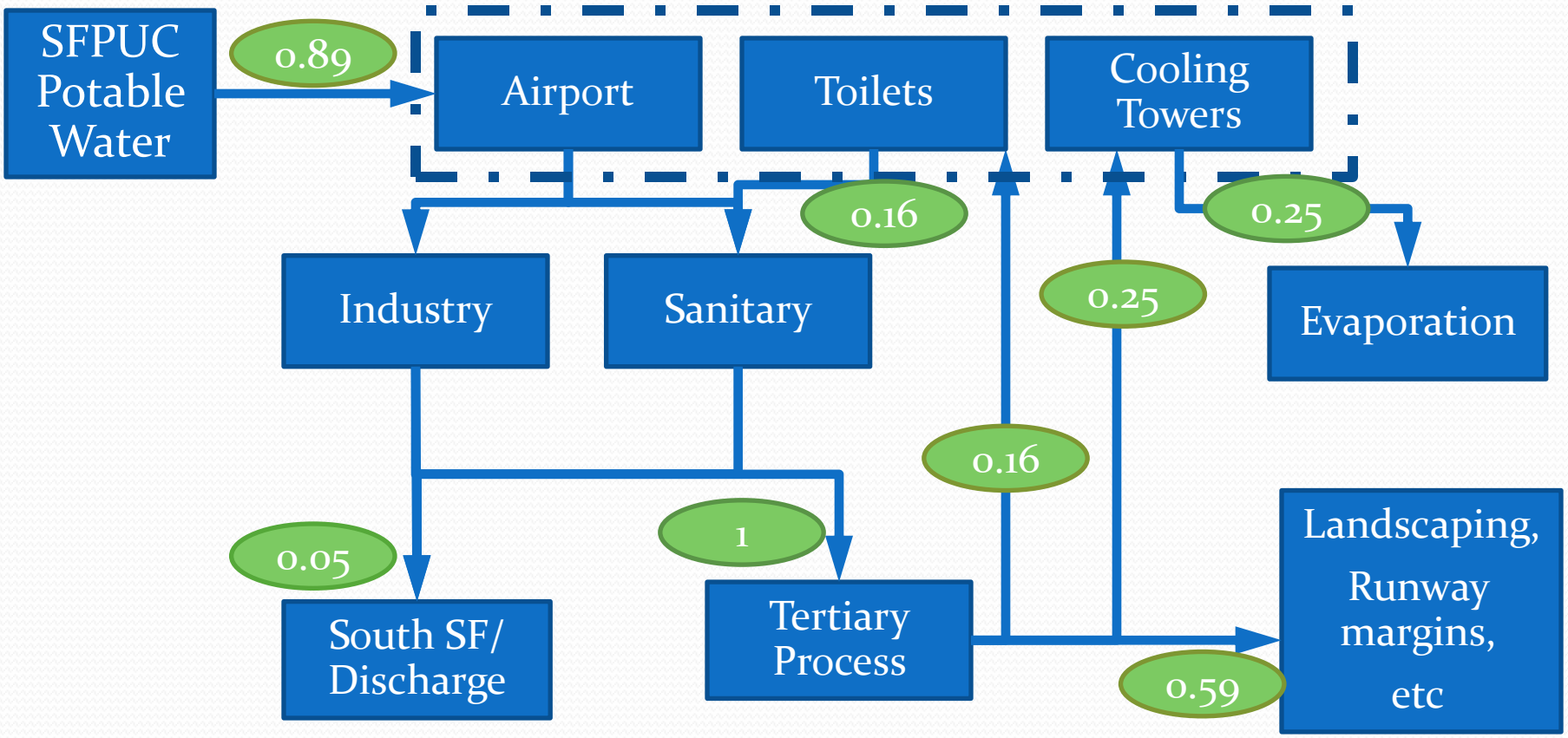
Process Flow Diagram-Current

All numbers in million gallons per day



Process Flow Diagram-New

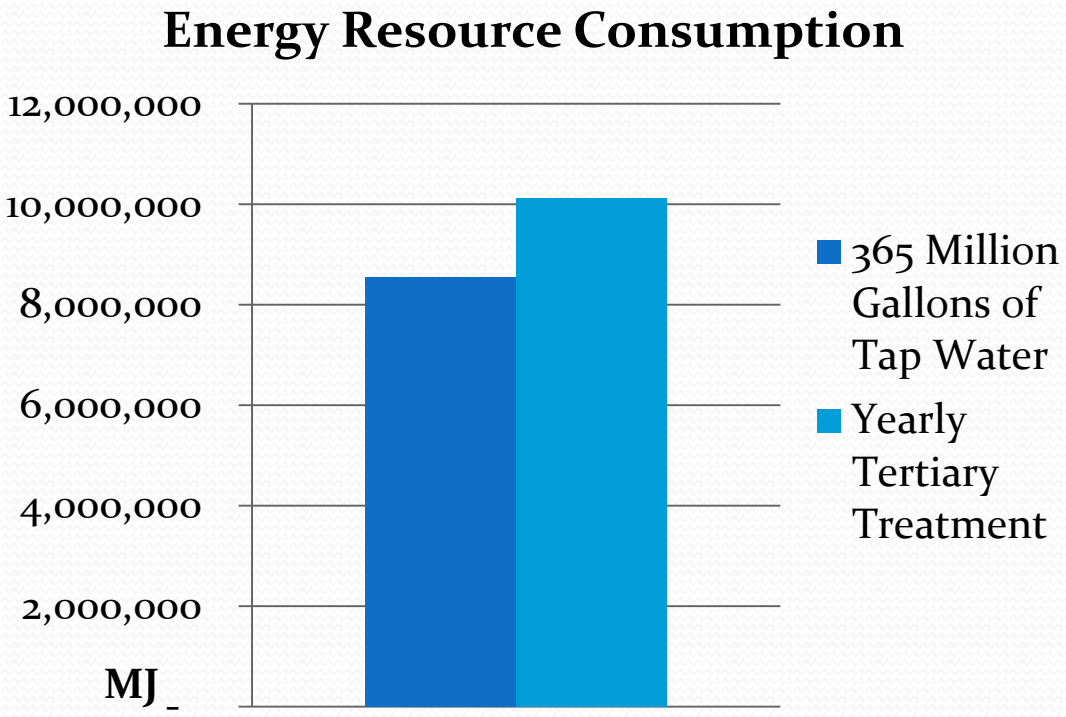
All numbers in million gallons per day



Energy Comparison

- Tertiary treatment results in an 18% increase in consumption of energy resources

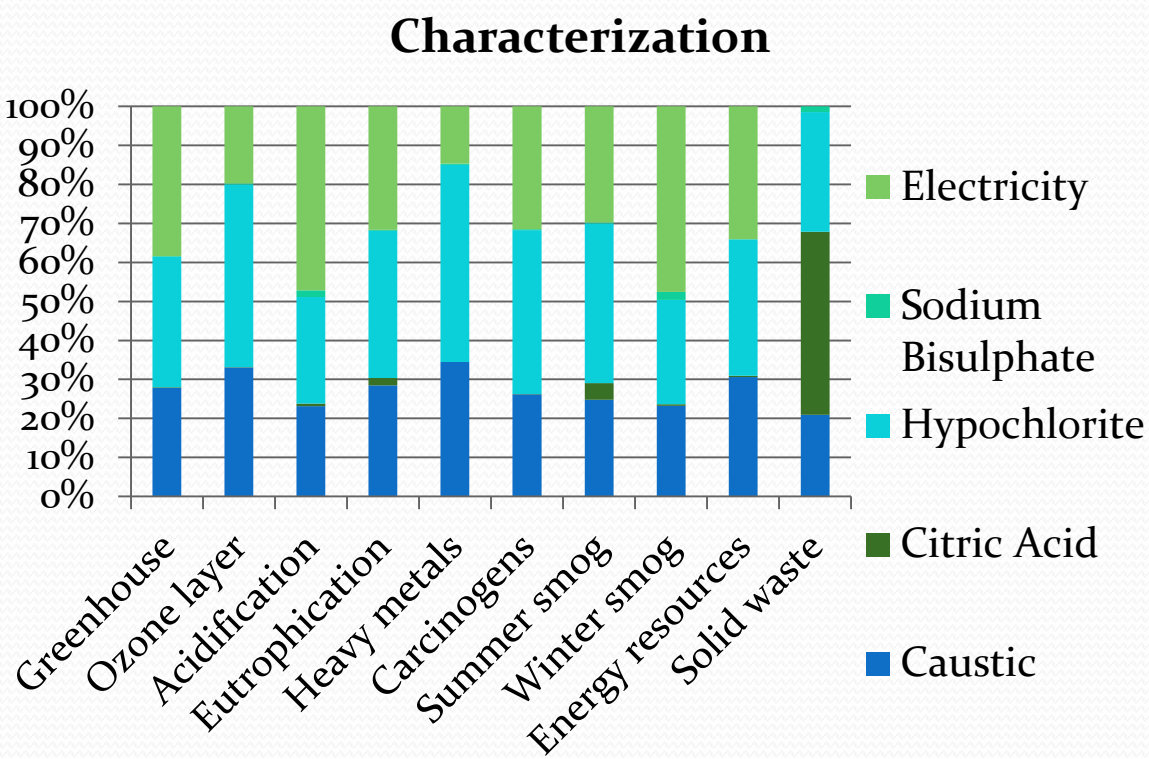
Total Increase	Percent Change
1,577,527 MJ	18%



Chemical Inputs

- **Caustic (Sodium Hydroxide)**
  - Added to raise the pH
  - 3,500 gallons per year of caustic in the new tertiary system.
- **Citric Acid**
  - Cleaning solution for membrane systems
  - 5,900 gallons of citric acid per year
- **Sodium Hypochlorite**
  - Disinfectant
  - 42,455 gallons per year
- **Sodium Bisulfate**
  - Added to the water to dechlorinate it
  - 2,400 gallons per year

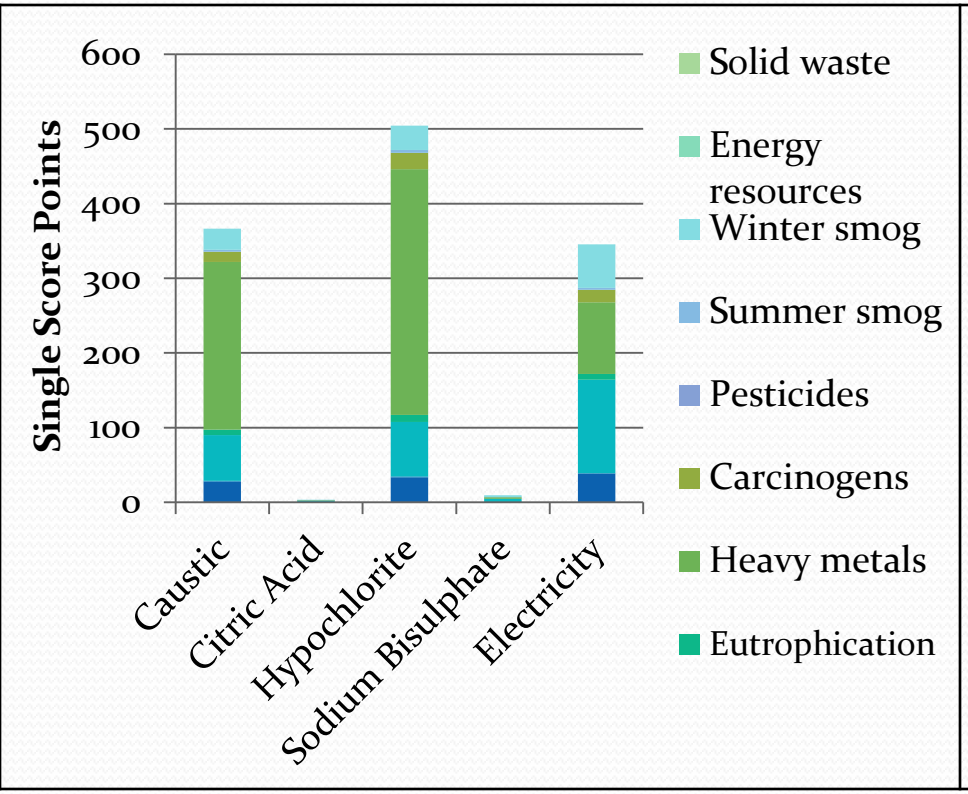
Pollutant Characterization

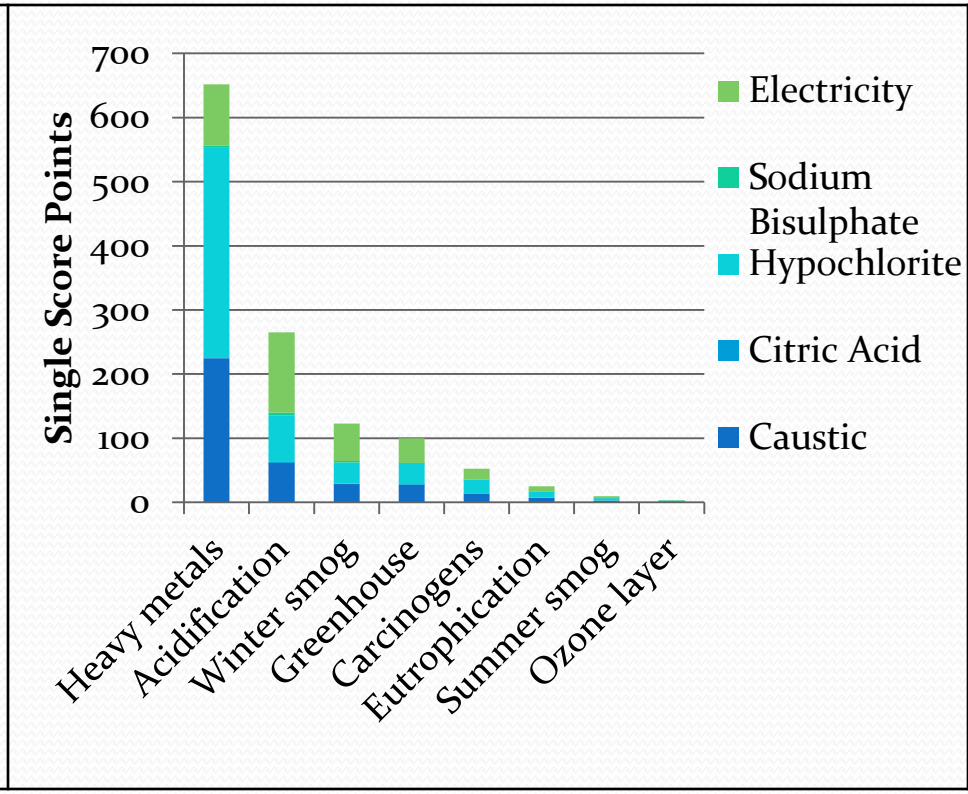


Ratios are largely consistent, with the exception of solid waste. Citric acid dominates solid waste due to the process of manufacturing it from beet sugar.

Environmental Impacts

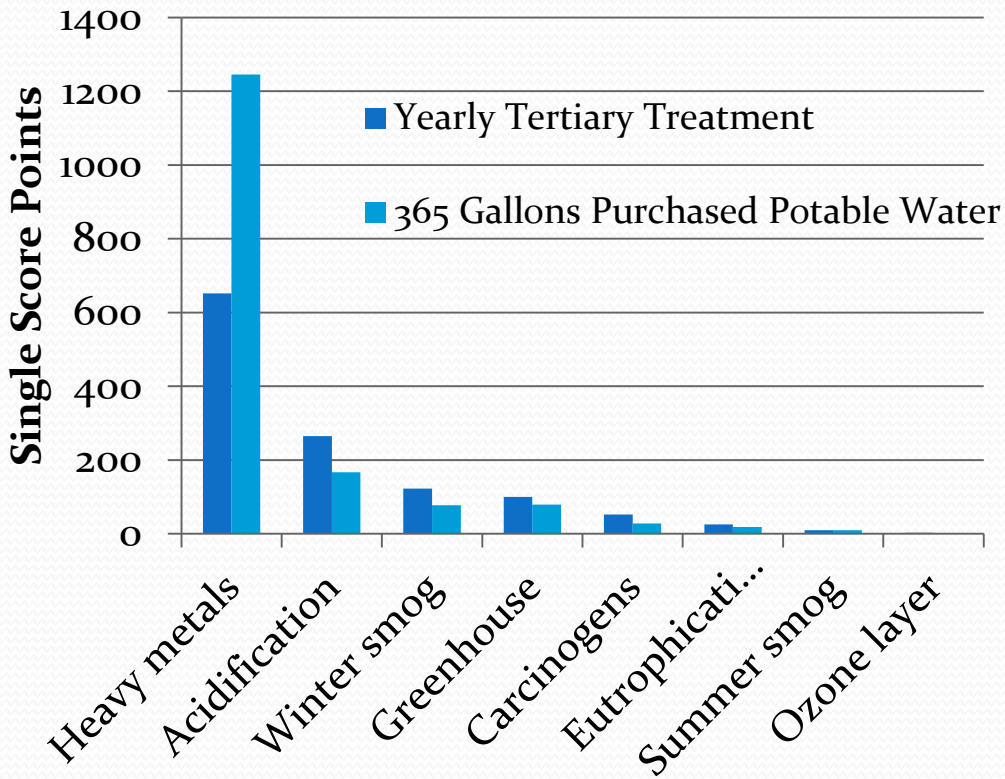
- Hypochlorite, electricity and caustic are most impactful
- In single score analysis, heavy metals are the most significant pollutants





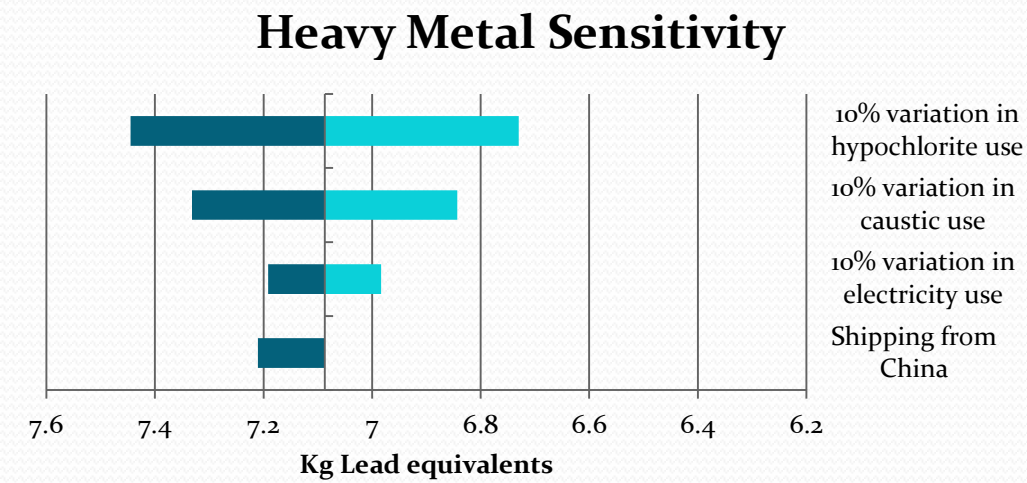
Pollutants Comparison

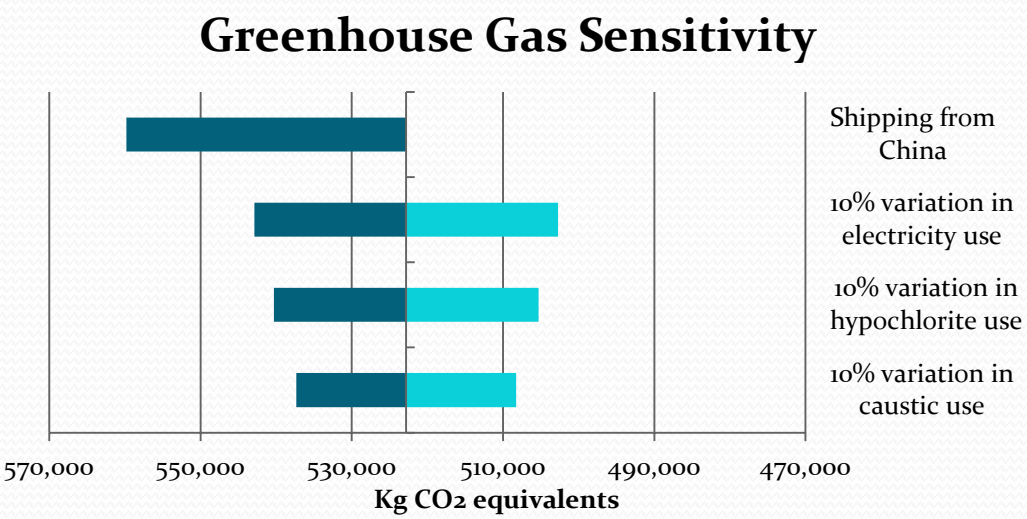
Tertiary treatment results in a 48% reduction in heavy metals pollution



Pollutant	Total Change	Percent Change
Heavy metals (kg lead)	-6.4	-48%
Acidification (kg SO_2)	1,108	59%
Winter smog (kg SPM)	849	58%
Greenhouse (tonne CO_2)	108	26%
Carcinogens (kg B(a)P)	0.03	85%
Eutrophication (kg PO_4)	54	40%
Summer smog (kg CH_4)	-3.4	-5%
Ozone layer (kg (kg CFC11)	0.01	26%

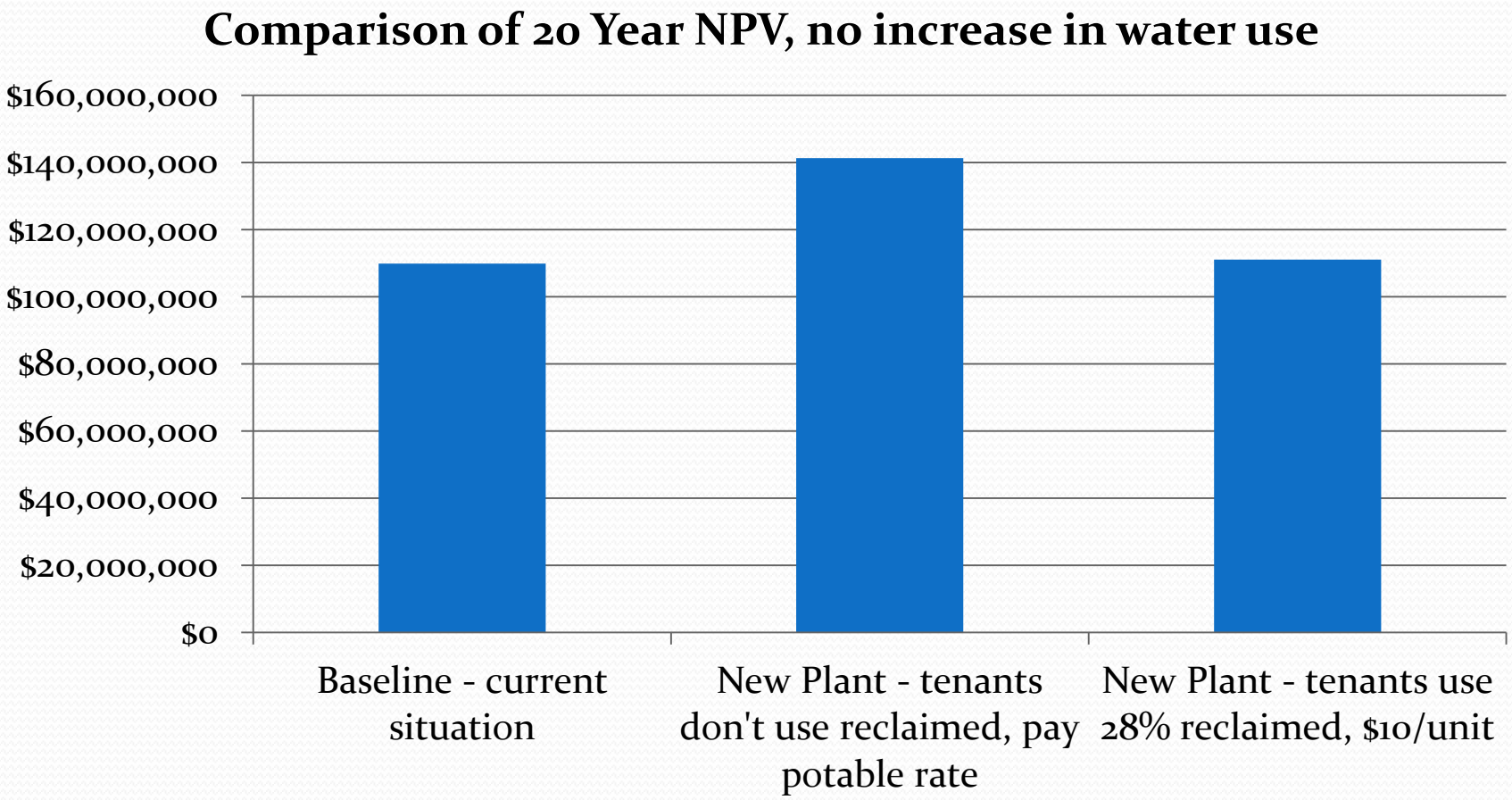
Environmental Sensitivity



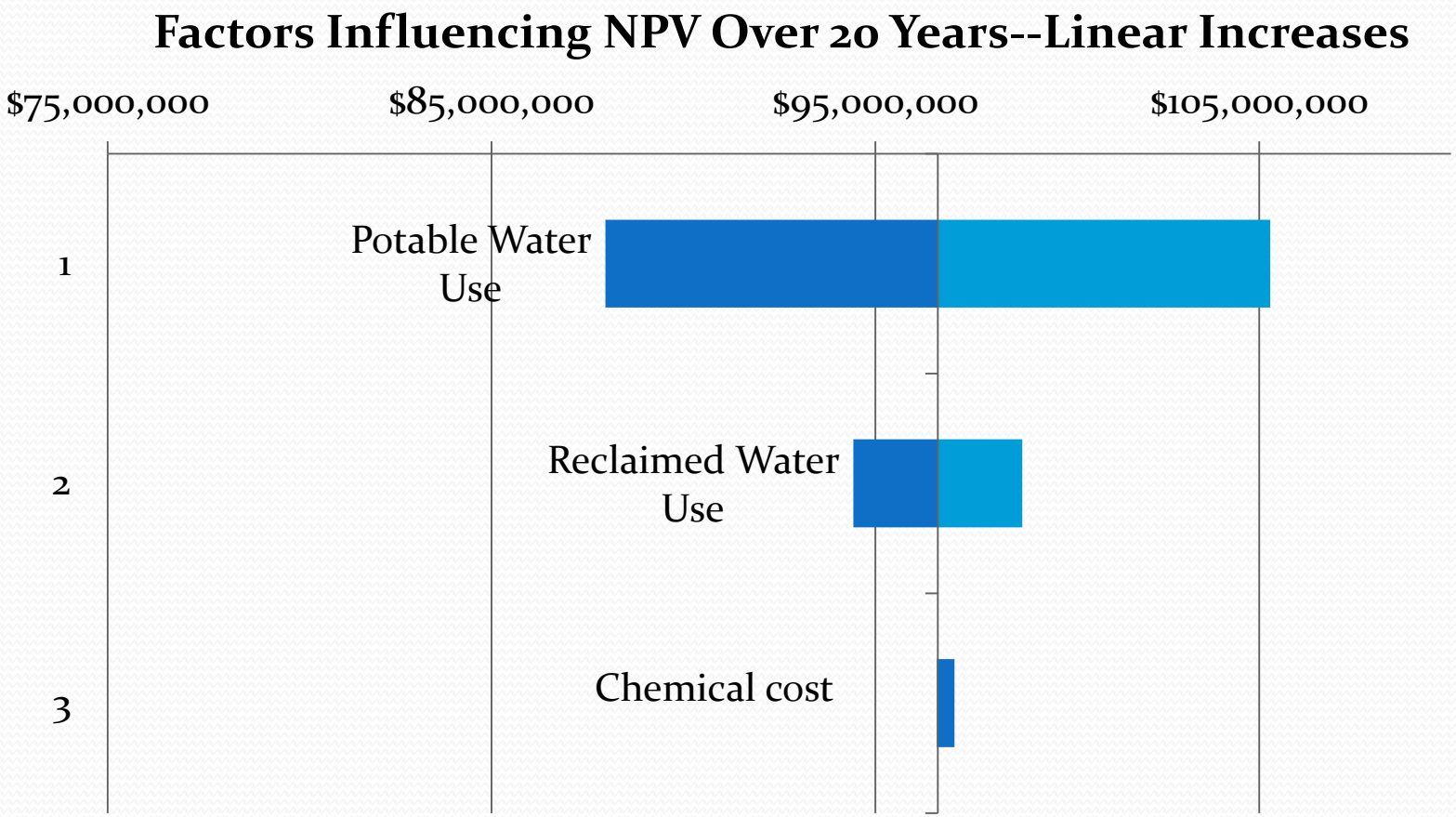


- Heavy metals
  - Focus on reducing hypochlorite and caustic consumption
- Green house gases
  - Focus on reducing chemical transportation distance and electricity consumption

Cost Comparison



Cost Sensitivity Analysis



Conclusion / Recommendations

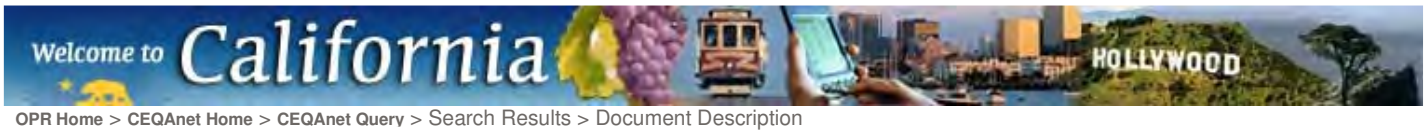
- Water use:
  - Less water from PUC and Hetch Hetchy
  - Have more uses for reclaimed water for additional savings
- Energy:
  - Slight increase in energy
- Environmental:
  - Small increase in most pollutants
  - ~50% reduction in heavy metals
  - Cut done on caustic use by change in wastewater policy
- Cost:
  - Tertiary plant is profitable
  - Charge tenant $10 per unit for reused water

Project 16

San José Green Street and Alleys Demonstration Project

Support Documents and References

- Martha Gardens CEQA Filing
- Spartan Keyes Neighborhood Improvement Plan Amendment
- NRDC Report: A Clear Blue Future Executive Summary
- San Mateo Green Streets Guidebook Introduction
- International BMP Database - Urban Stormwater BMP Monitoring Manual
- City of San José Public Works Guidance for Planning, Development, and Construction of Streetscape Projects
- City of San José Traffic Control Manual
- Envision San José 2040 General Plan
- Selbig and Bannerman (USGS Report 2007-5156)
- City of San José Public Works *Standard Details and Specifications*.
- EPA's Low Impact Development (LID) Literature Review
- Conceptual designs for the San José Green Streets & Alleys Demonstration Projects



Martha Gardens Green Alley Pilot Project

SCH Number: 2012018242

Document Type: NOE - Notice of Exemption

Project Lead Agency: San Jose, City of

Project Description

Martha Gardens Green Alley Pilot Project for a grant application to replace 35,000 square feet of existing pavement with green concrete (high recycle content, lighter colored concrete): to install four foot wide porous pavers and an infiltration trench along the middle of the alley: to install five drywells to capture excess water; and to replace eight existing street lights with LED lights. The project is a public improvement. No trees will be removed by the project.

Contact Information

Primary Contact:
Suzanne Thomas
City of San Jose
408 793 4386
70 West Hedding Street, East Wing
San Jose, CA 95110

Project Location

County: Santa Clara
City: San Jose
Region:
Cross Streets:
Latitude/Longitude:
Parcel No:
Township:
Range:
Section:
Base:
Other Location Info:

Exempt Status

- [ ] Ministerial
- [ ] Declared Emergency
- [ ] Emergency Project
- [x] Categorical Exemption
- [ ] Statutory Exemption

Type, Section or Code Number S:15301c,15302,15304(a)(f)

Reasons for Exemption

Existing Facilities, Replacement or Reconstruction and Minor Alterations to Land.

Date Received: 1/30/2012

County of Santa Clara
Office of the County Clerk-Recorder
Business Division

County Government Center
70 West Hedding Street, E. Wing, 1st Floor
San Jose, California 95110 (408) 299-5688

Santa Clara County Clerk–Recorder's Office
State of California

Document No.:	17204
Number of Pages:	4
Filed and Posted On:	1/27/2012
Through:	2/26/2012
CRO Order Number:	480831
Fee Total:	50.00

REGINA ALCOMENDRAS, County Clerk–Recorder
by Oscar Urquilla, Deputy Clerk–Recorder, OU

CEQA DOCUMENT DECLARATION

ENVIRONMENTAL FILING FEE RECEIPT

PLEASE COMPLETE THE FOLLOWING:

1. LEAD AGENCY: City of San Jose
2. PROJECT TITLE: Martha Gardens Green Alley Pilot Project, File No. PP12-011
3. APPLICANT NAME: City of San Jose, Dept of Environmental Services, Attn: Suzanne Thomas PHONE: (408) 793-4386
4. APPLICANT ADDRESS: 200 E. Santa Clara Street, Tower, 7th Floor, San Jose, CA 95113
5. PROJECT APPLICANT IS A: ☒ Local Public Agency ☐ School District ☐ Other Special District ☐ State Agency ☐ Private Entity
6. NOTICE TO BE POSTED FOR 30 DAYS.
7. CLASSIFICATION OF ENVIRONMENTAL DOCUMENT

a. PROJECTS THAT ARE SUBJECT TO DFG FEES

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☐ 2. NEGATIVE DECLARATION (PUBLIC RESOURCES CODE §21080(C)	$ 2,101.50	$ 0.00
☐ 3. APPLICATION FEE WATER DIVERSION (STATE WATER RESOURCES CONTROL BOARD ONLY)	$ 850.00	$ 0.00
☐ 4. PROJECTS SUBJECT TO CERTIFIED REGULATORY PROGRAMS	$ 992.50	$ 0.00
☐ 5. COUNTY ADMINISTRATIVE FEE (REQUIRED FOR a-1 THROUGH a-4 ABOVE) Fish & Game Code §711.4(e)	$ 50.00	$ 0.00

b. PROJECTS THAT ARE EXEMPT FROM DFG FEES

☒ 1. NOTICE OF EXEMPTION ($50.00 COUNTY ADMINISTRATIVE FEE REQUIRED)	$ 50.00	$ 50.00
☐ 2. A COMPLETED "CEQA FILING FEE NO EFFECT DETERMINATION FORM" FROM THE DEPARTMENT OF FISH & GAME, DOCUMENTING THE DFG'S DETERMINATION THAT THE PROJECT WILL HAVE NO EFFECT ON FISH, WILDLIFE AND HABITAT, OR AN OFFICIAL, DATED RECEIPT / PROOF OF PAYMENT SHOWING PREVIOUS PAYMENT OF THE DFG FILING FEE FOR THE *SAME PROJECT IS ATTACHED ($50.00 COUNTY ADMINISTRATIVE FEE REQUIRED)		
DOCUMENT TYPE: ☐ ENVIRONMENTAL IMPACT REPORT ☐ NEGATIVE DECLARATION	$ 50.00	$ 0.00

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☐ NOTICE OF PREPARATION ☐ NOTICE OF INTENT	NO FEE	$ NO FEE

8. OTHER: _____ FEE (IF APPLICABLE): $ _____
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CHECKS FOR ALL FEES SHOULD BE MADE PAYABLE TO: SANTA CLARA COUNTY CLERK-RECORDER

PLEASE NOTE: FEES ARE ANNUALLY ADJUSTED (Fish & Game Code §711.4(b); PLEASE CHECK WITH THIS OFFICE AND THE DEPARTMENT OF FISH AND GAME FOR THE LATEST FEE INFORMATION.

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CITY OF SAN JOSE
CAPITAL OF SILICON VALLEY

File#: 17204 1/27/2012

Department of Planning, Building and Code Enforcement
JOSEPH HORWEDEL, DIRECTOR

NOTICE OF EXEMPTION

To: County Clerk
County of Santa Clara County
70 West Hedding Street, East Wing
San Jose, California 95110
(408) 299-2481

From: Janis Moore, Planner II
Planning Implementation Division,
Environmental Review Section

Project File Number: PP12-011

Project Title: Martha Gardens Green Alley Pilot Project

Project Location, Address, Applicant: City Public Right-of-Way alley running parallel to and between S. 2nd and S. 3rd Streets, from Highway 280 to Martha Street, City of San Jose

Description of Nature, Purpose, and Beneficiaries of Project: Martha Gardens Green Alley Pilot Project for a grant application to replace 35,000 square feet of existing pavement with green concrete (high recycle content, lighter colored concrete); to install four foot wide porous pavers and an infiltration trench along the middle of the alley; to install five drywells to capture excess water; and to replace eight existing street lights with LED lights. The project is a public improvement. No trees will be removed by the project.

Name of Public Agency Approving Project: City of San Jose

Name of Person or Agency Implementing Project: City of San Jose, Dept of Environmental Services, Attn: Suzanne Thomas

Exempt Status: (check one)

- ☐ Ministerial (Sec. 21080(b)(1); 15268);
- ☐ Declared Emergency (Sec. 21080(b)(3); 15269(a));
- ☐ Emergency Project (Sec. 21080(b)(4); 15269(b)(c));
- ☒ Categorical Exemption. State type and section number: 15301(c), 15302, 15304(a)&(f) Existing Facilities, Replacement or Reconstruction and Minor Alterations to Land
- ☐ Statutory Exemptions. State code number: _____

Reasons why project is exempt: Project consists of minor alterations and improvements to an existing public right-of-way.

Lead Agency Contact Person: Suzanne Thomas

Area Code/Telephone /Extension: (408) 793-4386

If filed by applicant:

1. Attach certified document of exemption finding.
2. Has a Notice of Exemption been filed by the public agency approving the project? ☐ Yes ☐ No

Signature: Janis Moore Title: Planner II Date: January 26, 2012

☒ Signed by Lead Agency
☐ Signed by Applicant

Date received for filing at OPR: _____

CITY OF
SAN JOSE
CAPITAL OF SILICON VALLEY

Department of Planning, Building and Code Enforcement
JOSEPH HORWEDEL, DIRECTOR

STATEMENT OF EXEMPTION

FILE NO.	PP12-011
LOCATION OF PROPERTY	Alley right-of-way running parallel to and between S. 2nd and S. 3rd Streets, from Highway 280 to Martha Street
PROJECT DESCRIPTION	Martha Gardens Green Alley Pilot Project for a grant application to replace 35,000 square feet of existing pavement with green concrete (high recycled content, lighter colored concrete); to install four foot wide porous pavers and an infiltration trench along the middle of the alley; to install five drywells to capture excess water; and to replace eight existing street lights with LED lights. No trees will be removed by the project.
ASSESSOR'S PARCEL NUMBER	Public Right-of-Way

CERTIFICATION

Under the provisions of Section 15301(c), 15302, and 15304(a) and (f) of the State Guidelines for Implementation of the California Environmental Quality Act (CEQA) as stated below, this project is found to be exempt from the environmental review requirements of Title 21 of the San José Municipal Code, implementing the California Environmental Quality Act of 1970, as amended.

15301. Existing Facilities
Class 1 consists of the operation, repair, maintenance, permitting, leasing, licensing, or minor alteration of existing public or private structures, facilities, mechanical equipment, or topographical features, involving negligible or no expansion of use beyond that existing at the time of the lead agency's determination. The types of "existing facilities" itemized below are not intended to be all-inclusive of the types of projects which might fall within Class 1. The key consideration is whether the project involves negligible or no expansion of an existing use. Examples include but are not limited to:
(c) Existing highways and streets, sidewalks, gutters, bicycle and pedestrian trails, and similar facilities (this includes road grading for the purpose of public safety) except where the activity will involve removal of a scenic resource including a stand of trees, a rock out cropping, or an historic building.

15302. Replacement or Reconstruction
Class 2 consists of replacement or reconstruction of existing structures and facilities where the new structure will be located on the same site as the structure replaced and will have substantially the same purpose and capacity as the structure replaced.

15304. Minor Alterations to Land
Class 4 consists of minor public or private alterations in the condition of land, water, and/or vegetation which do not involve removal of healthy, mature, scenic trees except for forestry and agricultural purposes. Examples include but are not limited to:
(a) Grading on land with a slope of less than 10 percent, except that grading shall not be exempt in a waterway, in any wetland, in an officially designated (by federal, state, or local government action) scenic area, or in officially mapped areas of severe geologic hazard such as an Alquist-Priolo Earthquake Fault Zone or within an official Seismic Hazard Zone, as delineated by the State Geologist,
(f) Minor trenching and backfilling where the surface is restored.

Joseph Horwedel, Director
Planning, Building and Code Enforcement

Date: January 26, 2012

Deputy

Project Manager: Suzanne Thomas, ESD, Urban Runoff

Regina Alcomendras
Santa Clara Co.
Clerk-Recorder

01/27/12 16:04:06

Receipt # 2135581
Wrkstn ID C49A

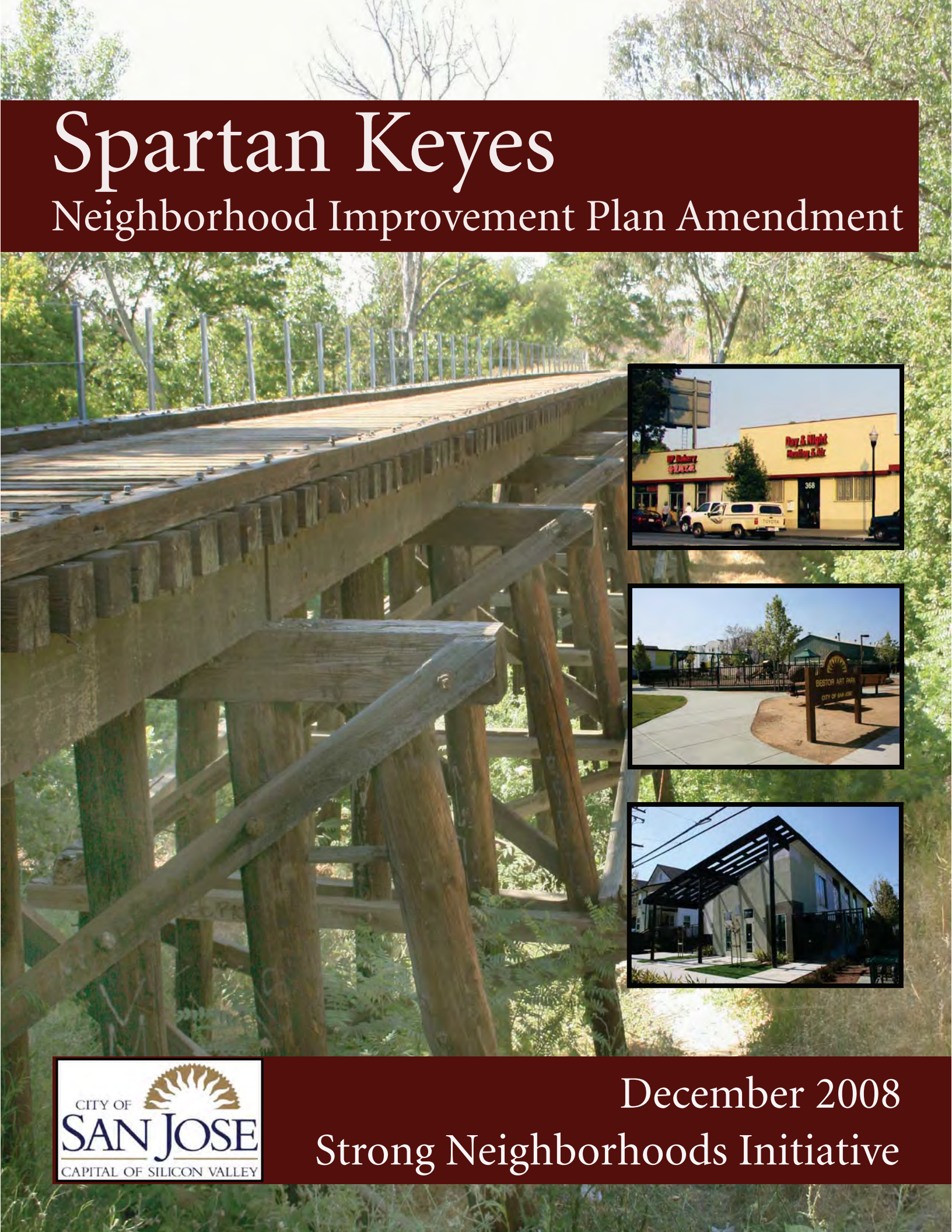
Public

Fees	$50.00
** TOTAL	50.00
Check	50.00

PLEASE KEEP

Spartan Keyes

Neighborhood Improvement Plan Amendment



CITY OF SAN JOSE
CAPITAL OF SILICON VALLEY

December 2008

Strong Neighborhoods Initiative

Strong Neighborhoods Initiative

Spartan Keyes Neighborhood Improvement Plan Amendment

City of San José
December 2, 2008

CITY OF
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2008 Action #10

Explore and, if possible, improve the condition of alleyways.

Two alleyways in poor condition in the Spartan Keyes SNI Area.

Issue

The condition of alleyways in the Spartan Keyes SNI Area has been an ongoing concern of residents. Some of these alleyways are unpaved, and others have deteriorating pavement, creating a very rough roadbed, as well as causing a lot of dust. Additionally, since all the alleyways lack drainage, water floods the roadbed during storms, as well as the backyards and parking garages of adjacent residences. Though many of the alleyways have some street lighting, a few do not. Community members feel that these conditions attract littering, graffiti, loitering, and criminal activities, such as drug and gang activity, and illegal dumping.

Discussion

Currently, many residents utilize the alleyways to access rear parking garages. Realizing it is not practical to close the alleyways, the Spartan Keyes community is focusing on improving the alleyways. All 10 alleyways in the Spartan Keyes SNI Area (see Figure 10 on page 50) needed to be upgraded.

Action Steps

a) *Examine conditions of the following ten alleyways (lettered A-J in Figure 10) in the Spartan Keyes SNI Area:*

- Alleyway A, between South 1st Street and South 2nd Street, from Margaret Street to East Virginia Street.
- Alleyway B, between South 1st Street and South 2nd Street, from East Virginia Street to Martha Street.
- Alleyway C, between South 2nd Street and South 3rd Street, from I-280 to Margaret Street.
- Alleyway D, between South 2nd Street and South 3rd Street, from Margaret Street to East Virginia Street.

- Alleyway E, between South 2nd Street and South 3rd Street, from East Virginia Street to Martha Street.
- Alleyway F, between South 5th Street and South 6th Street, from Keyes Street to East Humboldt Street.
- Alleyway G, between South 6th Street and South 7th Street, from Keyes Street northward toward Bestor Street (the alleyway does not go all the way through the block).
- Alleyway H, between South 6th Street and South 7th Street, from Keyes Street to East Humboldt Street.
- Alleyway I, between South 7th Street and South 8th Street, from Martha Street southward toward Keyes Street (the alleyway does not go all the way through the block).
- Alleyway J, between South 7th Street and South 8th Street, from Keyes Street northward toward Martha Street (the alleyway does not go all the way through the block).

Time Frame: Short-term (0-3 years)
Responsible Parties: NAC, DOT, and DPW staff

b) *Develop a capital improvement plan to upgrade the conditions of the alleyways in the Spartan Keyes SNI Area.*

Time Frame: Short-term (0-3 years)
Responsible Parties: DOT and DPW staff

c) *Improve the conditions of the alleyways, including the installation of new pavement, proper drainage, and streetlights where necessary.*

Time Frame: Medium-term to long-term (4-7+ years)
Responsible Parties: DOT and DPW staff

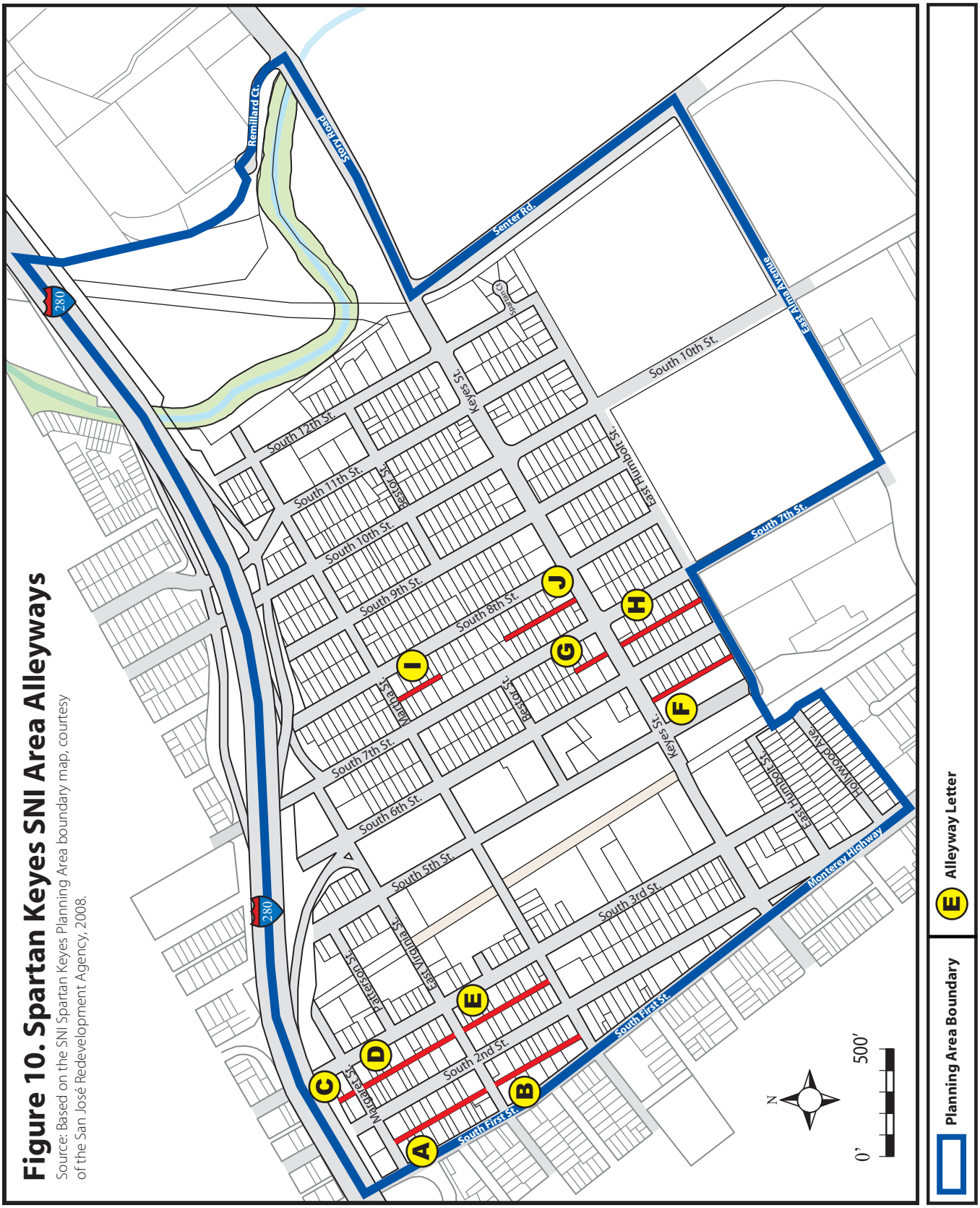

Figure 10. Spartan Keyes SNI Area Alleyways

Source: Based on the SNI Spartan Keyes Planning Area boundary map, courtesy of the San José Redevelopment Agency, 2008.

Appendix A

MATRIX OF TOP TEN ACTIONS AND ACTION STEPS

	Top Ten Action / Action Steps	Time Frame Immediate/Ongoing Short-term (0-3 yrs) Short- to Medium-term (0-6 yrs) Short- to Long-term (0-7+ yrs) Medium-term (4-6 yrs) Medium- to Long-term (4-7+ yrs) Long-term (7+ yrs)	Responsible Parties
1	**Redevelop the American Can Company site into a park, plaza, café, community/arts center, and various housing types, including artist studios.**	**S-L**	**NAC, PRNS, PBCE, SJRA**
	a) Create a vision plan for the American Can Company site and adjacent parcels for a park, plaza, café, community/arts center, and various housing types, including artist studios.	S	NAC, PRNS
	b) If necessary, amend the *Martha Gardens Specific Plan* to accommodate changes determined in the vision plan.	S	PBCE
	c) Work to attract, and then collaborate with, housing developers to redevelop the American Can Company site so that it includes a plaza, community/arts center, café, and various housing types, including artist studios.	M-L	NAC, SJRA
2	**Implement traffic calming strategies throughout the neighborhood.**	**S-L**	**DOT, SJSUPD, NAC**
	a) Slow and regulate traffic around SJSU Field during events.	S	SJSUPD
	b) Evaluate request to add a "no outlet" sign at East Humboldt Street. If the request is determined feasible, install sign.	S	DOT
	c) Evaluate and address safety concerns at the following intersections: Keyes Street/South 10th Street, Keyes Street/South 11th Street, East Humboldt Street/South 10th Street, and East Humboldt Street/South 11th Street.	S-M	DOT
	d) Evaluate and address traffic concerns on South 3rd Street, South 7th Street, South 10th Street, South 11th Street, Martha Street, Keyes Street, and East Humboldt Street. Focus efforts on streets near Spartan Stadium, especially along South 7th Street, between Keyes Street and East Alma Avenue.	S-M	DOT
	e) Convert the one-way couplet of South 2nd Street and South 3rd Street to two-way streets.	M-L	DOT
	f) Evaluate improving access to the Lowell Elementary School by improving the pedestrian safety of streets that pass under I-280.	S	DOT
	g) Improve the crosswalks at the South 7th Street/East Virginia Street intersection. Explore the feasibility of adding various traffic calming features such as bulb-outs to tighten turning radii, adding more pedestrian signage, and re-striping of crosswalks so they are more visible.	S	DOT
	h) Explore the possibility of integrating traffic calming signage with neighborhood welcome signage created by local artists.	S	NAC, DOT

MATRIX OF TOP TEN ACTIONS AND ACTION STEPS			
	Top Ten Action / Action Steps	**Time Frame**	**Responsible Parties**
3	**Create a neighborhood center with classes for adults and children to mingle. Increase services for children, and increase the involvement of Spartan Keyes residents in youth-based programs.**	**S-M**	**PRNS, SJRA, NAC**
	a) Pursue a lease for space to create a neighborhood center.	S	SJRA
	b) Use the center to provide youth services with assistance from volunteers and parents. Work to develop youth services, such as after school programs, homework clubs, and additional organized sports. Apply for grants to fund youth-based programs.	S	NAC, PRNS
	c) Identify non-general fund revenue sources for ongoing operation of the proposed neighborhood center.	S-M	NAC, PRNS
4	**Work with City staff to evaluate *General Plan* Land Use Designations. In particular, review high-density housing designations in established lower-density areas, where appropriate.**	**S**	**NAC, PBCE**
	a) Work with Planning staff to evaluate and, if appropriate, change *General Plan* Land Use Designations to be more in keeping with established neighborhood densities, particularly in the central and eastern portions of the Spartan Keyes SNI Area.	S	NAC, PBCE
	b) Work with Planning staff and applicants to ensure that new residential buildings are three stories in height or less, where appropriate.	S	NAC, PBCE
5	**Construct Coyote Creek Trail within the neighborhood.**	**S-L**	**PRNS, DOT, DPW**
	a) Prepare trail construction plans.	S-M	PRNS
	b) During the development of trail construction plans, explore the following: ▪ Aligning Coyote Creek Trail with the existing traffic signal at the Remillard Court/Story Road intersection to improve pedestrian safety. ▪ Creating a second trailhead at the Keyes Street/Senter Road intersection. ▪ Creating a connecting path from the Keyes Street/Senter Road intersection to Coyote Creek Trail.	S-M	DOT, PRNS
	c) Secure all necessary funding for the trail.	S-L	PRNS
	d) Acquire the abandoned Union Pacific Railroad ROW through Story Road Landfill, to allow for a connection from Coyote Creek Trail to Keyes Street via the existing rail bridge, and to permit development of the Five Wounds Trail.	S-L	DPW, PRNS
	e) Construct the trail.	M-L	PRNS

MATRIX OF TOP TEN ACTIONS AND ACTION STEPS			
	Top Ten Action / Action Steps	**Time Frame**	**Responsible Parties**
6	**Improve safety in the neighborhood.**	**I/O**	**SJPD, NAC, Art Ark Management**
	a) Work with the SJPD to strengthen the relationship between residents and officers.	I/O	NAC, SJPD
	b) Work with SJPD to provide periodic enforcement of illegal activities at the homeless encampments near Coyote Creek and south of the Cash and Carry building.	I/O	SJPD
	c) Improve security at the Art Ark Apartments.	I/O	NAC, Art Ark Management
7	**Provide more code enforcement in the neighborhood, targeting blight created by abandoned houses and vehicles.**	**I/O**	**PBCE, NAC, SNI**
	a) Ensure that NAC meetings are attended quarterly by code enforcement staff.	I/O	PBCE
	b) Encourage NAC members and other community members to participate in the SNI Blight Busters Program.	I/O	NAC, PBCE, SNI
8	**Provide a new neighborhood park. In particular, explore utilizing the land to the west and south of the Cash and Carry lot for a park.**	**S-M**	**PRNS, DPW**
	a) Explore the feasibility of utilizing land to the west and south of the Cash and Carry lot for use as a park.	S	DPW, PRNS
	b) If feasible, create a master plan for the park.	S-M	PRNS
	c) Secure necessary funding for the park.	S-M	PRNS
	d) Prepare construction plans and build the park.	M	PRNS
9	**Attract businesses that meet the needs of residents, including a café with outdoor seating, medium-sized grocery store, convenience store, drug store, and a post office.**	**S-M**	**NAC, PBCE, SJRA**
	a) Explore and, if feasible, amend the *General Plan* by adding a NBD overlay designation along Keyes Street from South 1st Street to Senter Road.	S	NAC, PBCE, SJRA
	b) Encourage the establishment of a BIA along with a BID for businesses along Keyes Street.	S-M	NAC, SJRA
	c) Look for opportunities to include desired services in new housing developments.	S	NAC, SJRA
	d) Explore potential sites in the Spartan Keyes SNI Area for desired services.	S	NAC, SJRA

MATRIX OF TOP TEN ACTIONS AND ACTION STEPS		
Top Ten Action / Action Steps	**Time Frame**	**Responsible Parties**
10 Explore and, if possible, improve the conditions of alleyways.	**S-L**	**NAC, DOT, DPW**
a) Examine conditions of the following ten alleyways in the Spartan Keyes SNI Area: ▪ Alleyway A, between South 1st Street and South 2nd Street, from Margaret Street to East Virginia Street. ▪ Alleyway B, between South 1st Street and South 2nd Street, from East Virginia Street to Martha Street. ▪ Alleyway C, between South 2nd Street and South 3rd Street, from I-280 to Margaret Street. ▪ Alleyway D, between South 2nd Street and South 3rd Street, from Margaret Street to East Virginia Street. ▪ Alleyway E, between South 2nd Street and South 3rd Street, from East Virginia Street to Martha Street. ▪ Alleyway F, between South 5th Street and South 6th Street, from Keyes Street to East Humboldt Street. ▪ Alleyway G, between South 6th Street and South 7th Street, from Keyes Street northward toward Bestor Street (the alleyway does not go all the way through the block). ▪ Alleyway H, between South 6th Street and South 7th Street, from Keyes Street to East Humboldt Street. ▪ Alleyway I, between South 7th Street and South 8th Street, from Martha Street southward toward Keyes Street (the alleyway does not go all the way through the block). ▪ Alleyway J, between South 7th Street and South 8th Street, from Keyes Street northward toward Martha Street (the alleyway does not go all the way through the block).	S	NAC, DOT, DPW
b) Develop a capital improvement plan to upgrade the conditions of the alleyways in the Spartan Keyes SNI Area.	S	DOT, DPW
c) Improve the conditions of the alleyways, including the installation of new pavement, proper drainage, and streetlights, where necessary.	M-L	DOT, DPW

Appendix B – Glossary of Acronyms

2002 *NIP*	2002 *Spartan Keyes Neighborhood Improvement Plan*
2008 *NIPA*	2008 *Spartan Keyes Neighborhood Improvement Plan Amendment*
BIA	Business Improvement Association
BID	Business Improvement District
DOT	Department of Transportation
DPW	Department of Public Works
LUTD	Land Use/Transportation Diagram
MGPC	Martha Gardens Planned Community
NAC	Spartan Keyes Neighborhood Action Coalition
NBD	Neighborhood Business District
PBCE	Department of Planning, Building, and Code Enforcement
PRNS	Department of Parks, Recreation, and Neighborhood Services
RAA	Renewing the Action Agenda
ROW	Right-of-Way
SAFETEA-LU	Safe Accountable Flexible Efficient Transportation Equity Act: A Legacy for Users
SJPD	San José Police Department
SJRA	San José Redevelopment Agency
SJSU	San José State University
SJSUPD	San José State University Police Department
SNI	Strong Neighborhoods Initiative
VTA	Valley Transportation Authority

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Appendix C – Reference List

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A Clear Blue Future

How Greening California Cities Can Address Water Resources and Climate Challenges in the 21st Century

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UNIVERSITY OF CALIFORNIA, SANTA BARBARA

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The mission of the Donald Bren School of Environmental Science and Management, University of California, Santa Barbara, is to play a leading role in researching environmental issues, identifying and solving environmental problems, and training research scientists and environmental management professionals.

Acknowledgments

This report received substantial input and review from a number of individuals. Their participation contributed greatly to the quality of the report and its conclusions and recommendations. We acknowledge that they may not necessarily endorse all of the reports recommendations. In particular, we wish to thank our peer reviewers: Martha Davis, Inland Empire Utilities Agency; Lillian Kawasaki, Water Replenishmenr District of Southern California; and Ted Johnson, Water Replenishment District of Southern California. We also wish to thank Paula Daniels, City of Los Angeles; Haan-Fawn Chau, City of Los Angeles; Fran Spivy-Weber, California State Water Resources Control Board; Caren Trgovcich, California State Water Resources Control Board; Bob Languell, California Air Resources Board; Aubrey Dugger, GreenInfo Network; Stacy Tellinghuisen, Wester Resource Advocates; Michael Antos, Los Angeles and San Gabriel Rivers Watershed Council; Edward Belden, Los Angeles and San Gabriel Rivers Watershed Council; Mark Wildermuth, Wildermuth Environmental, Inc.; Doug Pushard, American Rainwater Carchment Systems Association; Jack Schultz, American Rainwater Catchment Systems Association; Dr. Tamim Younos, Virginia Water Resources Research Center, Virginia Tech; Rohb Whitaker, Water Replenishment District of Southern California; David Nahai, Los Angeles Department of Water and Power; James McDaniel, Los Angeles Department of Water and Power; James Yannotta, Los Angeles Department of Water and Power; Nancy Sutley, Council on Environmental Quality; Dr. Arturo Keller, UC Santa Barbara; Fiona Sanchez, Irvine Ranch Water District; Brad Cross, LFR, Inc. In addition, NRDC would like to thank The Summit Foundation and Environment Now for their generous support of our work.

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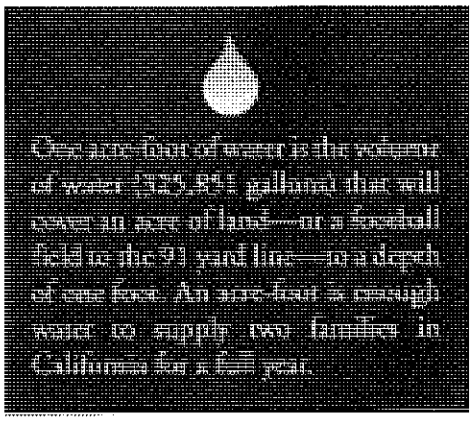
Executive Summary

As global warming threatens our water resources, communities are faced with a need to respond quickly and economically to water supply shortfalls. Both the snowpack and surface runoff that form a critical supply of potable water for western states are being affected by higher temperatures. Low impact development, or LID, is a land planning and engineering design approach to stormwater management that enables cities, states, and individuals to increase access to safe and reliable sources of water while reducing the amount of energy consumed and global warming pollution generated by supplying the water. New NRDC and UCSB analysis shows that implementing LID practices at new and redeveloped residential and commercial properties in parts of California can increase water supplies by billions of gallons each year, providing an effective and much-needed way to mitigate global warming's impact on California.

LID Techniques Can Deliver Water and Energy Savings for Californians

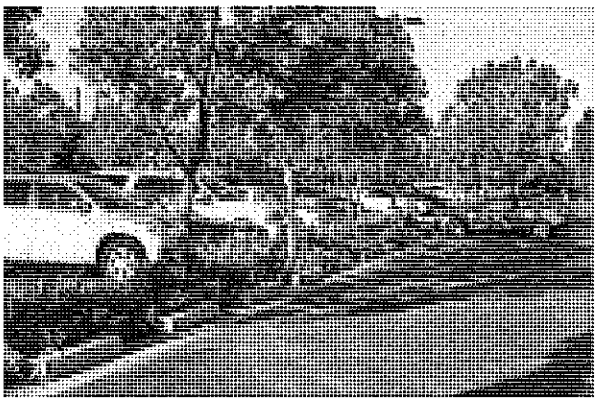
The NRDC and UCSB analysis found that implementing LID practices that emphasize rainwater harvesting, which includes infiltration of water into the ground as well as capture in rain barrels or cisterns for later use onsite, at new and redeveloped residential and commercial properties in the urbanized areas of southern California and limited portions of the San Francisco Bay area has the potential to increase local water supplies by up to 405,000 acre-feet (af) of water per year by 2030. This volume represents roughly two-thirds of the volume of water used by the entire City of Los Angeles each year. The water savings translate into electricity savings of up to 1,225,500 megawatt hours (MWh), avoiding the release of as much as 535,500 metric tons of CO_2 per year, as the increase in energy-efficient local water supply from LID results in a decrease in the need to obtain water from imported sources of water such as the California State Water Project (SWP) or the use of processes such as ocean desalination, both of which require tremendous amounts of energy. These benefits would increase in each year thereafter.



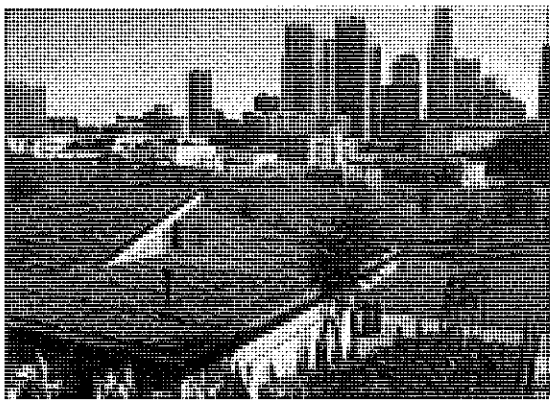
And the true value of LID is likely much higher. Our analysis currently assumes a conservative figure for future development rates and does not account for the loss of water that currently occurs as it is conveyed from distant sources. Expanding the use of LID to industrial, government, public use, and transportation development and redevelopment in southern California has the potential to yield an additional 75,000 acre-feet of savings per year by 2030, with corresponding reductions in energy use and CO_2 emissions.

Moreover, even greater overall water and electricity savings—and associated reductions in global warming pollution—would result from full application of LID practices statewide and in other areas of the United States where augmenting local water supplies may reduce the amount of energy required to supply water from more energy-intensive sources.

The 1,223,500 megawatt hours of electricity savings that can be achieved each year through use of LID practices represents enough energy to power more than 102,000 single family homes for one full year. Emissions reductions of 333,960 metric tons of CO_2 each year are the equivalent of taking more than 57,000 cars off the road. The analysis we present in this paper shows that LID can play a significant role in issues of addressing areas of water supply and climate change throughout California and the southwest United States.



Vegetated swale in a parking lot CREDIT: HAAN-FAWN CHAU



Green roof in Vista Hermosa Park, Santa Monica Mountains Conservancy, Los Angeles / CREDIT: Ken Weston and Reza Iranpour/ City of Los Angeles

LID Technologies Provide Multiple Benefits

LID was developed to ameliorate—and where possible, eliminate—the pollution and erosion problems generated by runoff from urban and suburban development at the source, where rain falls on paved surfaces, by maximizing the natural onsite infiltration and treatment abilities of soils and vegetation or by capturing water for later use. It provides important environmental benefits by reducing pollution of downstream rivers, lakes, and coastal waters.

Successful LID practices include:

- maximizing infiltration, which recharges local and regional groundwater systems;
- providing retention areas and slowing runoff, which reduce flooding and erosion;
- minimizing the impervious footprint of a project through reducing paved surfaces;
- directing runoff from impervious areas onto landscaping; and
- capturing runoff in rain barrels or cisterns for beneficial use.

By preventing site runoff altogether in many situations, LID is substantially more effective at protecting water quality than many types of conventional water management practices that rely on structural treatment devices to remove a percentage of pollution after it has already entered stormwater runoff. In addition to serving as a superior method of stormwater pollution control, LID can increase water supply reliability in a region prone to natural disasters, serve to reduce flooding and erosion associated with urban runoff, reduce the "heat island" effect from solar radiation in urban settings, and provide green space and open land, enhancing property values. The use of LID can also reduce the costs of municipal stormwater infrastructure and decrease the frequency and severity of combined sewer overflow events.

LID Is Cost Effective

The U.S. Environmental Protection Agency (EPA) states that "LID practices can reduce project costs and improve environmental performance" of development and that, with few exceptions, LID has been "shown to be both fiscally and environmentally beneficial to communities."[3] As a result, LID practices can provide a targeted, cost-effective means of addressing issues of water pollution, water supply, and climate change all at once.

Current and Emerging Regulatory Policies Support the Implementation of LID

Federal and state regulatory policies already require that developed sites in larger and midsized cities control post-construction stormwater runoff. Requiring implementation of LID technologies can therefore reduce energy use and global warming pollution and serve as a cost-effective means of complying with existing mandates of federal and state laws.

Groundwater's Role in California's Water Supply

By allowing more water to infiltrate the ground and recharge aquifers, LID can reduce demand for energy-intensive imported water or desalinated ocean water. Recharging these aquifers is particularly useful given that most areas of the state already have infrastructure in place to extract and distribute groundwater. California extracts more groundwater—approximately 17 million acre-feet per year—than any other state in the country. As much as 50 percent of the state's population receives some portion of their potable water supply from groundwater, including the vast majority of urbanized southern California areas that also receive a portion of their water from energy-intensive sources in northern California. In fact, nearly 50 percent of the total population of the United States depends on groundwater for some part of their water supply, meaning that LID practices can be used to help recharge local and regional groundwater aquifers across large portions of the country.

The reliance on groundwater is particularly strong in southern California where an average of 1.56 million acre-feet per year—or about 40 percent of the region's total water needs—are met through local groundwater pumping. Rainwater is the primary source that recharges the aquifers, and the Metropolitan Water District of Southern California recently estimated that ground water basins in the southern California region have 3.2 million acre-feet of storage space available for possible recharge. This existing capacity underlines the potential for LID practices that emphasize infiltration to greatly enhance local groundwater supplies. The use of LID represents a practical solution to California's water supply needs.

Capturing Rooftop Runoff

For areas where surface soil conditions, aquifer capacity, or traditional water supply patterns may favor capturing runoff rather than groundwater recharge, practices that promote capture and beneficial use provide a similar opportunity to reduce energy use and greenhouse gas (GHG) emissions. LID practices for capturing rainfall can reduce runoff volumes by as much as 75 percent, saving much of the water to be used onsite.

The Climate Challenge

Global warming is already affecting water resources in California, and temperatures are projected to rise by as much as 8 to 10.5 degrees Fahrenheit in the state toward the end of the century. The Sierra snowpack, which forms California's largest freshwater surface reservoir and serves as a critical source of drinking and agricultural irrigation water, is expected to shrink

by between 25 and 40 percent by 2050, and as much as 70 to 90 percent by the end of the century, drastically increasing the strain on a water management system that will be needed to supply an additional 24 million people by midcentury. Reservoirs along the Colorado River, which supply large portions of the southern California population, have slowly emptied to less than half their capacity as drought conditions grip the region. California must look for fresh and innovative ways to address climate change and reduce the emission of greenhouse gases that threaten its water supply.

The safe and sufficient supply of water in California is both a casualty of global warming and a contributor to it. Water must often be conveyed long distances and at great energy cost that results in substantial GHG emissions. In fact, water now constitutes one of the largest uses of energy in California, consuming an astonishing 19 percent of all electricity and 33 percent of non-power-plant natural gas used in the state. A significant portion of the electricity, substantially more than the national average, is used for the conveyance of water. The California State Water Project (SWP), which pumps water a distance of 444 miles from the Sacramento-San Joaquin Delta to southern California, and lifts the water from just above sea level at the Delta nearly 3,000 feet over the Tehachapi Mountains in the process, is the single-largest individual user of electricity in the state. And as California confronts issues related to limited water supplies and a growing economy, 20 ocean desalination plants have been proposed statewide, each of which would supply water at an energy cost comparable to conveying water through the SWP. By contrast, the energy required to supply groundwater can be 5 to 20 times less than that required to supply water through the SWP or ocean desalination, and the energy required for capture and onsite use of stormwater can be 8 to more than 25 times less—if there are any energy requirements at all. LID presents a way to augment local water supply and avoid the energy consumption and GHG emissions associated with transporting water over long distances.

The white "bathtub rings" show the pre-drought water level of Lake Powell in Arizona. In March 2008, the lake's water levels had fallen more than 100 feet below its corresponding levels a decade ago.

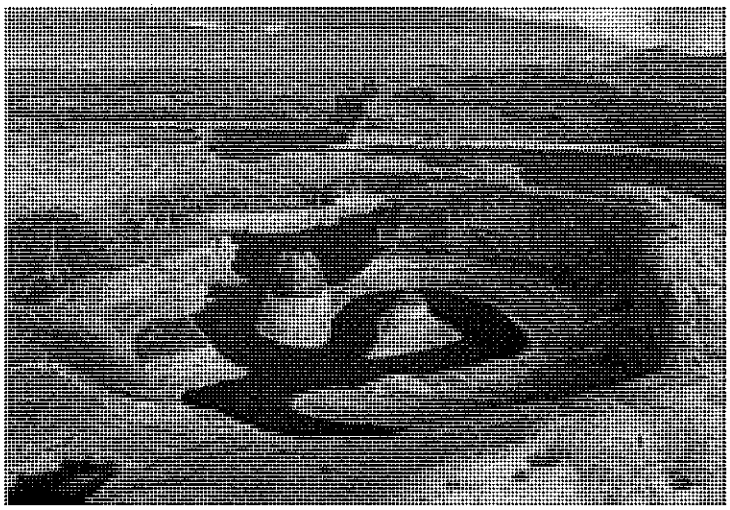


PHOTO CREDIT: Mike Reyfman, *Lake Powell.*

The LID Solution

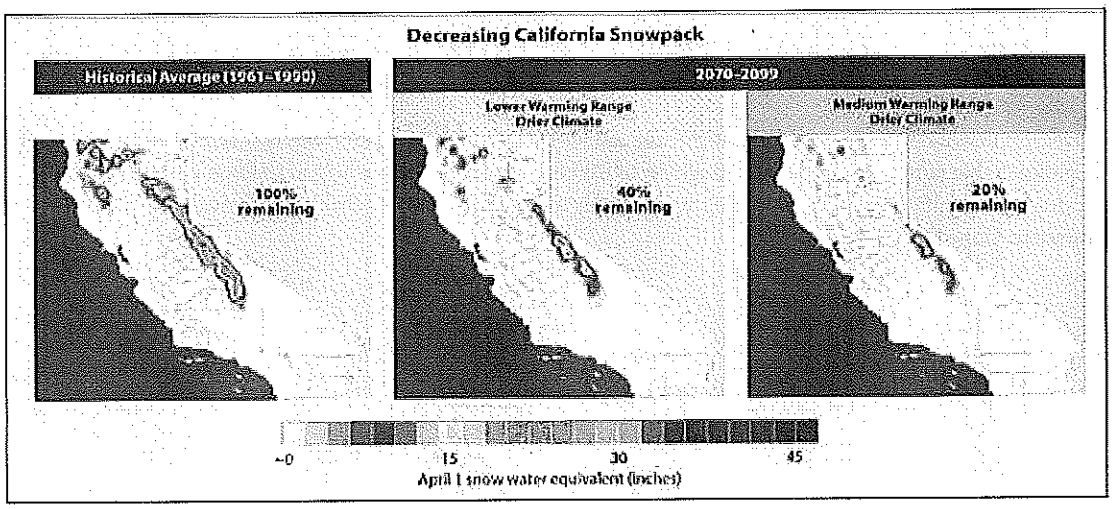
LID provides new ways to adapt to and mitigate the serious challenges to our water supply posed by global warming. Our study indicates that rainwater harvesting through use of LID can be of considerable help to California—and at bargain prices. LID can play a significant role in addressing water supply and global warming challenges throughout California and the southwestern United States.

GLOBAL WARMING

Global warming is already affecting water resources throughout the western United States. The California Department of Water Resources (DWR) states that, although "the exact conditions of future climate change remain uncertain, there is no doubt about the changes that have already happened."[1] According to DWR, "[t]he average early spring snowpack in the Sierra Nevada decreased by about 10 percent during the last century, a loss of 1.5 million acre-feet of snowpack storage."[2] Forming the largest freshwater reservoir in California, the Sierra snowpack is a critical source of water for the entire state. "Snowmelt currently provides an annual average of 15 million acre-feet of water, slowly released between April and July each year. Much of the state's water infrastructure was designed to capture the slow spring runoff and deliver it during the drier summer and fall months."[3] However, changes as a result of global warming threaten the continued viability of this vital water source. Largely because of temperature increase, the Sierra snowpack is projected to shrink by between 25 to 40 percent by 2050, and as much as 70 to 90 percent by the end of the century (see Figure 1).[4,5]

The rise in temperatures in the Sierra Nevada Mountains has two major implications for California's water supply. First, "more precipitation is falling as rain and less as snow."[6] An air temperature increase of 1.8°F (1°C) is predicted to reduce the average annual volume of water produced from snowmelt by approximately 15 percent, while a 7.2°F (4°C) increase would result in about a 60 percent reduction in snowmelt.[7] Second, snowmelt is occurring progressively earlier in the season.[8] In the early spring, man-made reservoirs in California are operated for flood control purposes. Water must be released from the reservoirs, rather than stored, to protect against the possibility of storms or heavy precipitation events late in the wet season. As snowmelt and the resulting runoff shifts to earlier times in the year, it reduces the amount of water

Figure 1: Projected dry-climate reduction in the Sierra Nevada snowpack, from 2070–2099; from the California Climate Change Center, 2006. Projections are based on warming ranges of 3–5.5°F (1.7-3.3°C) (Lower Warming Range) and 5.5–8°F (3-4.4°C) (Medium Warming Range). The High Warming Range estimate, which projects a temperature increase of 8 to 10.5°F (4.4 to 5.8°C), is not shown.



SOURCE: California Climate Change Center.

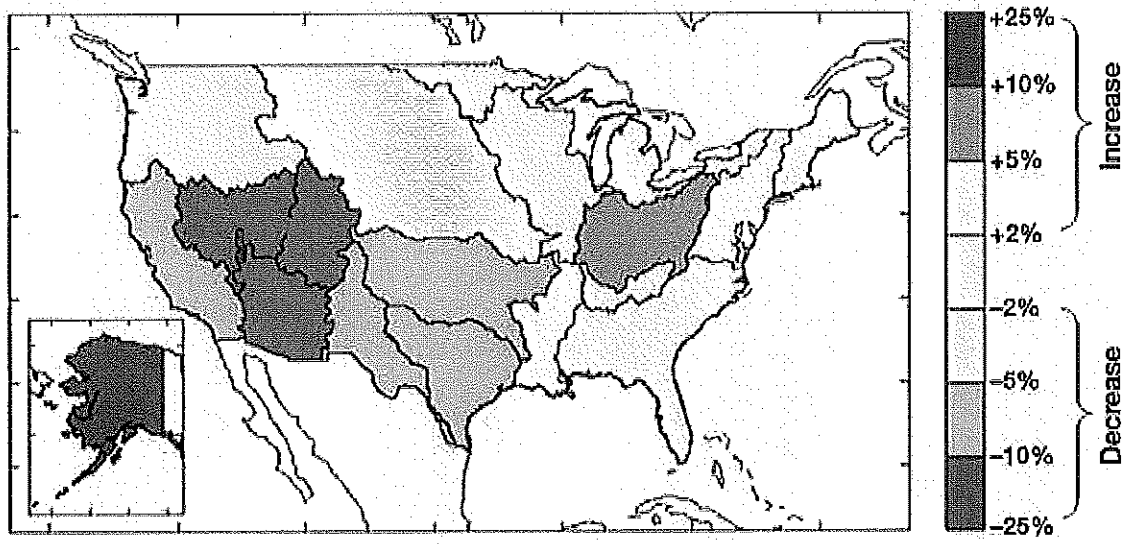
AND WATER

available for storage in California's water supply system. An increase in the average temperature of 7.2°F (4°C) would shift the mean snowmelt runoff from mid-March to mid-February, when reservoirs must still release water. Thus, even if the same volume of snowmelt were to occur in the future, less of the water could be stored and later made available for use during the dry California summer.[9]

Global warming is predicted to have a significant impact on surface streamflow and runoff as a result of the increased temperatures and altered precipitation patterns. The overall surface runoff in California could decrease by up to 10 percent by midcenrury, with far greater decreases in runoff in the intermountain Southwesr (see Figure 2).[10] With a rise in temperatures of between 5.5 and 8°F (3 to 4.4°C) by the century's end, the medium warming range predicted by the California Climate Change Center, late spring streamflow could drop by as much as 30 percent.[11]

These changes come in addition to projected increases in sea level rise that threaten coastal aquifers with saltwater intrusion, an increase in the number and intensity of extreme storm events thar cause erosion and flooding, warmer remperatures that increase evapotranspiration rates and therefore the amount of water needed for irrigation of crops and landscaping, and increased warer pollution tesulting from rising temperatures and decreased streamflow.[12] There is an urgent need to both manage water and reduce GHG emissions, and LID offers a potential avenue for addressing these concerns. In addition to providing GHG emissions reductions (known as "mitigation" in climate discussions), LID also provides potentially valuable adaptation attributes such as improved water supply reliability and water quality.

Figure 2. Projected changes in runoff for the period of 2041–2060 relative to 1901–1970. Modified from U.S. Climate Change Science Program, 2008.



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CHAPTER 1

INTRODUCTION TO THE GUIDEBOOK

Project Summary

In 2004 the California State Legislature approved and the Governor enacted Assembly Bill 1546 that authorized the City/County Association of Governments of San Mateo County to impose a vehicle registration fee on motor vehicles registered in the county for the management of traffic congestion and stormwater pollution. A portion of the funds raised by this fee are earmarked for projects that help mitigate the environmental impact of automobiles on the environment, particularly stormwater runoff from roadways and parking lots.

SOURCE: HTTP://WEB.NESTUCCA.K12.OR.US.TEC.TEC.HTM

Figure 1-1: *Site-level design begins with the first drops of rain.*

The San Mateo Countywide Water Pollution Prevention Program (SMCWPPP) initiated two measures to address the detrimental impacts of stormwater runoff from transportation infrastructure on County waters. The first measure involved creating a grant program using vehicle registration fees to fund the construction of demonstration projects in the County. The demonstration projects showcase the use of Best Management Practices (BMPs) to reduce, capture and treat runoff from roadways and parking lots. In 2008, C/CAG awarded demonstration project grant funds to five San Mateo County municipalities and approved a second call for projects in FY 2008/09 to fund additional demonstration projects. It is intended that these demonstration projects provide inspiration for future sustainable stormwater projects in the San Mateo County region.

The second phase was the production of this guidebook in order to provide the immediate need in assisting San Mateo County municipalities with designing green street and parking lot demonstration projects. This guidebook is intended to inspire small but widespread changes that will improve San Mateo County's watershed health. Conceptual design examples from some of the FY 2008 grant-funded demonstration projects are also included in the guidebook.

The Sustainable Green Streets and Parking Lots Guidebook is to be used in conjunction with the SMCWPPP's *C.3 Stormwater Technical Guidance: A Handbook for Developers, Builders and Project Applicants* (C.3 Technical Guidance). As the name suggests, that document is focused on providing step-by-step technical guidance on how to design, permit, and maintain post-construction stormwater controls in order to meet the current stormwater management requirements mandated in Provision C.3 of the countywide municipal stormwater permit. The primary purpose of this guidebook is to help the user identify and realize green street and parking lot site design opportunities, provide solutions to common implementation barriers, and provide guidance on how to best design, construct, and maintain successful projects. Any technical information provided herein is superceded by equivalent information in the C.3 Technical Guidance.

SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-2: *The conventional approach to stormwater management is treating rainfall runoff as a waste rather than a resource.*

SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-3: *Sustainable stormwater design strives for a more natural, cost effective, and visible approach to managing runoff.*

SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-4: *The Sustainable Stormwater Design Model. A balance of economy, ecology, and society.*

For much of the last century, drainage systems have been engineered to quickly collect runoff in underground pipes and carry it away using an "out of sight, out of mind" approach. This design philosophy treats rainfall runoff as a waste, and many people are unaware of the stormwater flowing in pipes underneath city streets when it rains.

Sustainable stormwater design treats rainfall runoff as a valuable resource. It is based on balancing urban development while preserving natural hydrological functions. Furthermore, sustainable stormwater design achieves the multiple goals of being cost effective, improving water quality, and addressing community concerns. Mimicking the natural hydrologic function of healthy ecosystems in street and parking lot landscapes can dramatically reduce pollution, decrease runoff volume, reduce runoff temperature, protect aquatic habitat, and create more interesting places to live.

The following pages illustrate how the natural environment functions prior to urban development, the overall effects of creating impervious area, and methods of redesigning urban landscapes to help bring healthy hydrological functions back into our neighborhoods.

SUSTAINABLE STORMWATER DESIGN PRINCIPLES

1. Manage stormwater at the source and on the surface. As soon as rainfall lands on a street or parking lot, allow it to infiltrate into the ground or provide surface flow to nearby landscaping.

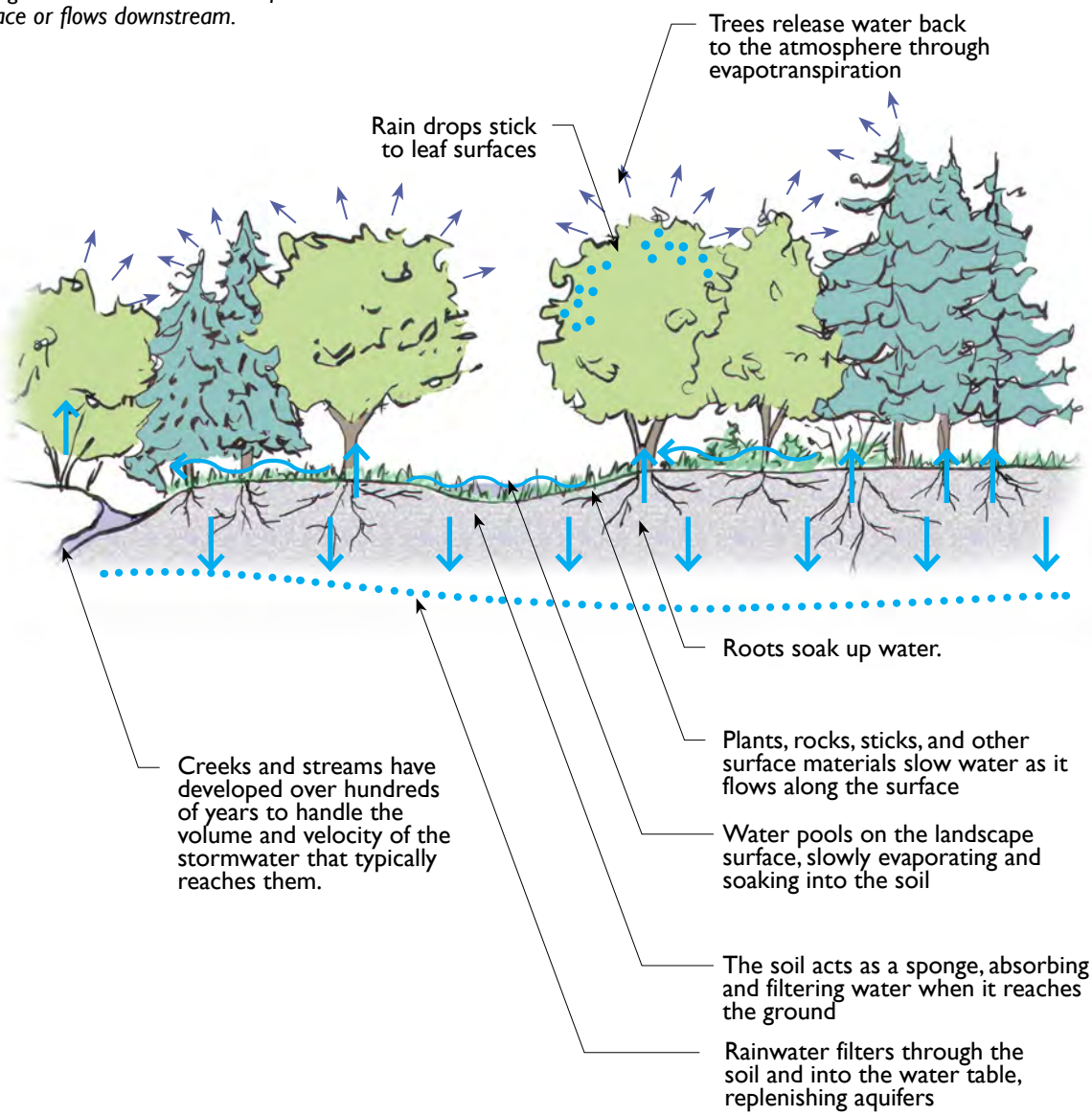
2. Use plants and soil to absorb, slow, filter, and cleanse runoff. Let nature do its work.

3. Design stormwater facilities that are simple, cost-effective and enhance community aesthetics. Stormwater facilities can be beautiful!

SOURCE: WWW.FLICKR.COM - DBARON

Figure 1-5: *A thick layer of trees, shrubs, groundcovers, and grasses absorbs water before it reaches the soil surface or flows downstream.*

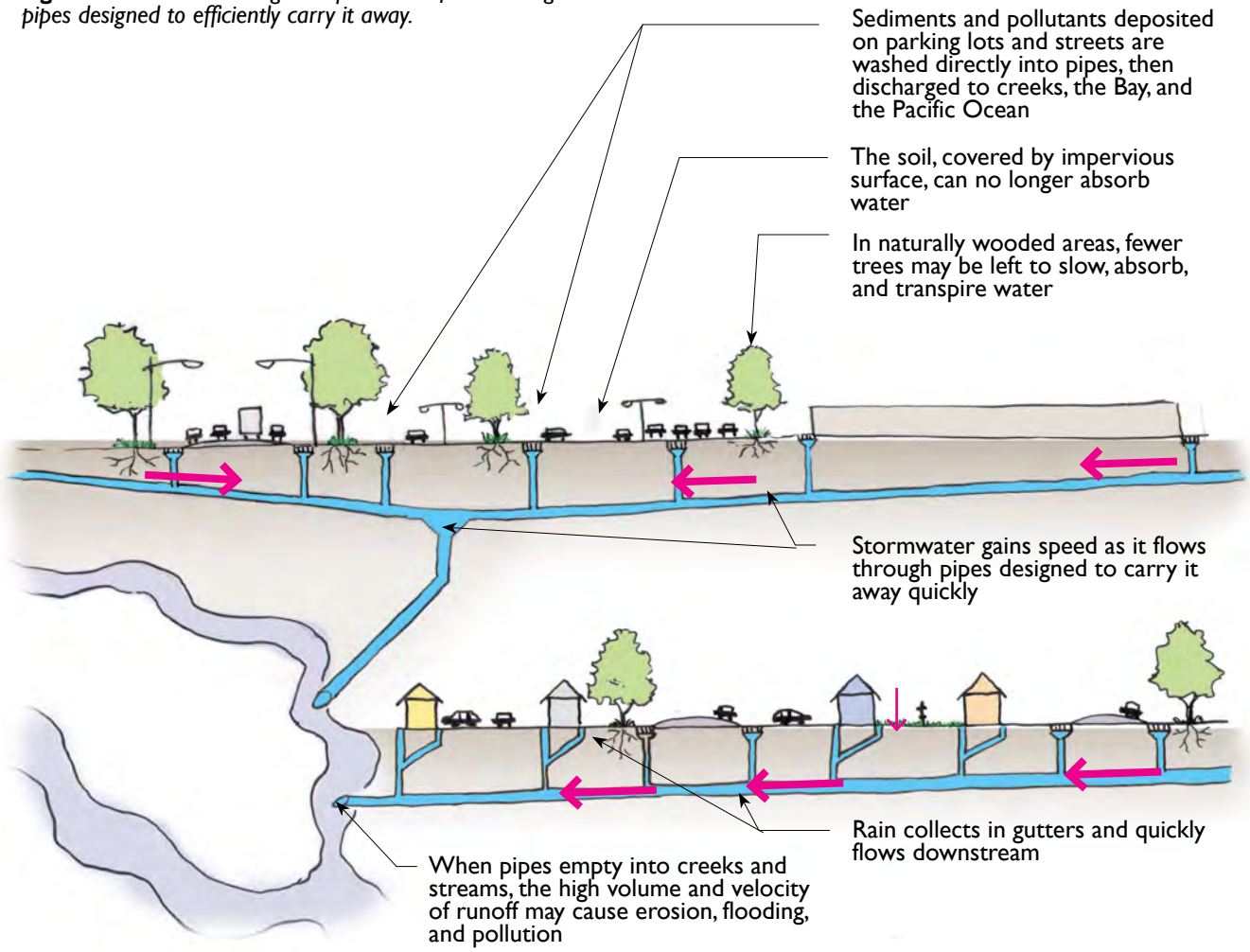
A healthy, undisturbed landscape acts like a sponge by capturing, absorbing, and slowing the flow of water from the moment a raindrop lands on the ground. Urban development, though, has dramatically impacted natural hydrologic systems by reducing the landscape's absorptive capacity and introducing pollutants.



SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-6: *Stormwater gains speed as it flows through pipes designed to efficiently carry it away.*

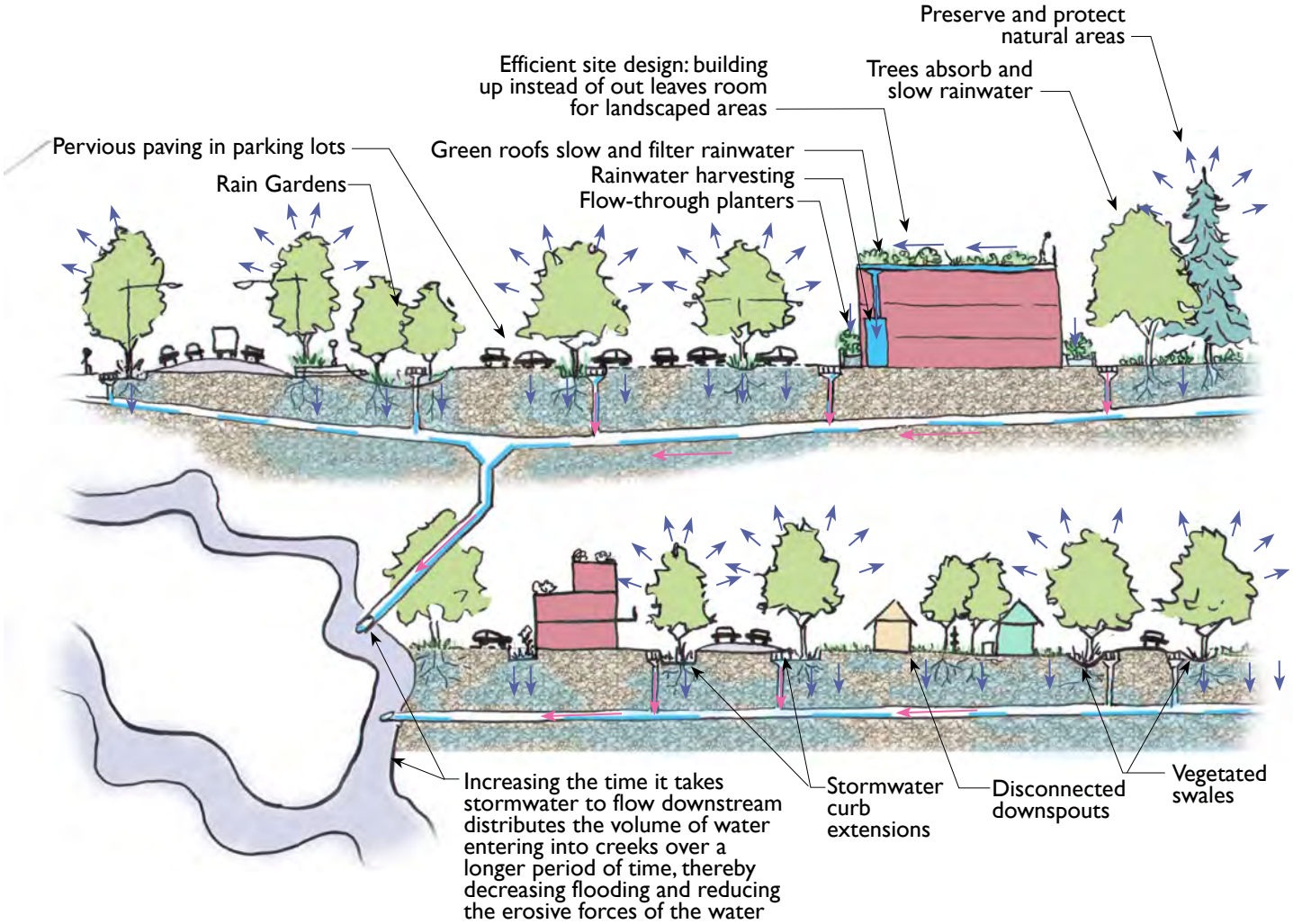
When the natural landscape is urbanized, impervious surface is created that prevents water from being absorbed at the source. Sediments and pollutants from streets, parking lots, homes, yards, and other sources are washed into pipes and water bodies. Stormwater runoff increases as more and more impervious surface is created. The high volume and velocity of stormwater runoff emptying into creeks and streams may cause flooding and erosion, destroying natural habitat. There is a better approach.



Infrastructure can be designed to minimize its impact on natural drainage systems. Our infrastructure can help maintain the balance of natural drainage systems by capturing, slowing, and absorbing stormwater, as well as filtering the pollutants that urban development introduces.

SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-7: *Infrastructure can help protect creeks and streams by capturing, slowing, and absorbing stormwater and filtering pollutants.*



SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-8: *Stormwater facilities filter sediments and other pollutants in runoff; which results in improved water quality.*

SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-9: *Stormwater facilities slow the flow of stormwater runoff through the interaction of the water with plants and soil.*

SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-10: *Stormwater facilities collect and absorb stormwater to reduce the overall volume of runoff.*

The Three Stormwater Management Goals

Sustainable stormwater design should achieve the following three goals to the greatest extent possible:

Water Quality Goal

Stormwater facilities should filter and **remove** excess sediments and other pollutants from runoff. By allowing water to interact with plants and soil, water quality improvements are achieved through a variety of natural physical and chemical processes. Even if soils are not conducive to infiltration, or if there is a high water table, water quality is still enhanced through pollutant settling, absorption into the soil, and uptake by plants.

Flow Reduction Goal

Stormwater facilities should **slow** the velocity of runoff by detaining stormwater in the landscape. Flow rate reduction can often be achieved by integrating design strategies (such as pervious paving, planter boxes, swales, and rain gardens) that provide stormwater detention. By detaining and delaying runoff, peak flow rates are attenuated and downstream creeks are protected from erosive flows. Conveying runoff through a system of naturalized surface features mimics the natural hydrological cycle and minimizes the need for underground drainage infrastructure.

Volume Reduction Goal

Whenever possible, facilities should collect and **absorb** stormwater to reduce the overall volume of runoff. Retention facilities offer long-term stormwater collection and storage for reuse or groundwater recharge. Plants contribute to retention capacity by intercepting rainfall, taking up water from the soil, and assisting infiltration by maintaining soil porosity. Volume reduction does not require stormwater facilities to be extremely deep. In fact, it is usually best to employ a highly integrated and interconnected system of shallow stormwater facilities.

There is a lot of variability in how a "green street" or "green parking lot" is defined. For the purposes of this guidebook, they include streets and parking lots designed with a landscape and/or paving system that captures, slows, filters, and potentially infiltrates stormwater runoff. Green streets and parking lots provide stormwater reduction and water quality benefits to runoff before discharging to local creeks. Specific design strategies are discussed in detail in Chapter 2.

Figure 1-11 below describes different levels of green design based on how aggressively a particular site manages runoff. For example, a street or parking lot with substantial landscape areas and a system of broad canopy trees to capture rainfall is a Level 2 design, even though it has no dedicated stormwater treatment measures. On days with minimal rainfall, a majority of the rainfall may be captured within the tree structure and ground landscaping.

However, green streets and parking lots are most commonly thought of as introducing some type of stormwater treatment measure (e.g., vegetated swale, planter, rain garden, etc.) to actively capture and manage surface runoff at its source. This is a Level 3 design and represents the most common perception of a green street or parking lot. But green streets can move beyond a Level 3 design.

Green Streets and Parking Lots are designed with a landscape element and/or pervious pavement system that captures, slows, filters, and potentially infiltrates stormwater runoff into the ground.

The concepts of livability and stormwater management are intertwined for Level 4 and 5 designs and are primarily related to green streets rather than parking lots. A Level 4 green street not only encompasses the attributes of Levels 1, 2, and 3, but also provides a direct emphasis

Figure 1-11:
GREEN STREETS AND PARKING LOTS CAN BE "MULTIPLE SHADES OF GREEN"

Level 1 Maximizes landscape areas along the street and minimizes overall impervious areas of the land. Some runoff from sidewalks may be managed in landscape areas.

Level 2 Significant tree canopy is added to the urban streetscape.

Level 3 Fully manages street, sidewalk, and driveway runoff by using a landscape system. Design solutions are cost effective, provide direct environmental benefits, and are aesthetically pleasing.

Level 4 Green street provides direct focus on alternative modes of transportation including mass transit, biking, and walking.

Level 5 Green street frontage manages both public and private stormwater runoff. Building, site, and street frontage become one integrated space designed for stormwater management.

SOURCE: NEVUE NGAN ASSOCIATES

on alternative transportation options, such as walking, biking, and/or using mass transit. More people using alternative transportation lessens the number of vehicles generating pollution. Furthermore, incorporating mass transit stops, bike lanes and racks, carpool drop off areas, or other similar site design measures can reduce the overall impervious area required.

The "greenest," and most difficult level to achieve, is a Level 5 design. This comprehensive approach allows stormwater to be managed within the entire street "envelope," which blurs the line between public and private space. Stormwater from private driveways and buildings could be managed within the public right-of-way. Conversely, stormwater from the street could utilize available landscape space within private property. This is currently not a widely-accepted condition here in the United States; however, in many European cities, this type of a green street is becoming more common.

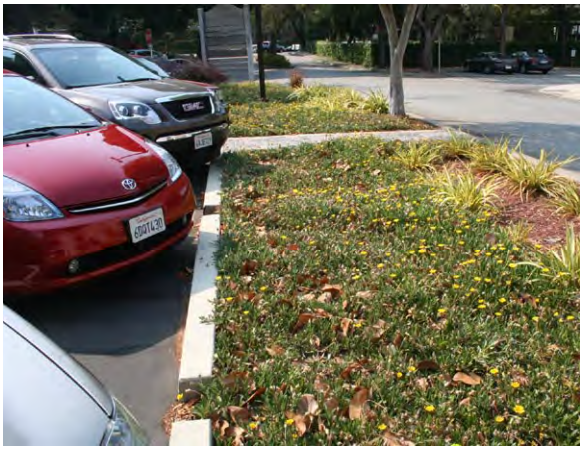
New and redevelopment projects offer more opportunities to achieve a Level 4 or 5 design. Other projects (especially retrofits), due to a multitude of site constraints, might only be able to achieve a Level 3 design. Regardless, the most important consideration is to always strive to reach the highest level of green design possible. When a high level of green design is applied to street and parking lot sites throughout the County, the overall health of the watershed, the San Francisco Bay, and the Pacific Ocean will improve.

SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-12: *Green streets can be urban in form.*

SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-13: *Green infrastructure can also be very free-form and artistic.*



SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-14: *A vegetated swale within a green parking lot in San Mateo County.*

SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-15: *When it rains on our streets, pollutants are washed directly into pipes and then into creeks, the Bay, or the Pacific Ocean.*

SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-16: *Even small parking lots contribute to the larger problem of increased stormwater runoff.*

SOURCE: CITYWIDE FACILITIES IMPROVEMENTS, STORM DRAIN IMPROVEMENTS REPORT, CITY OF BURLINGAME, 2004

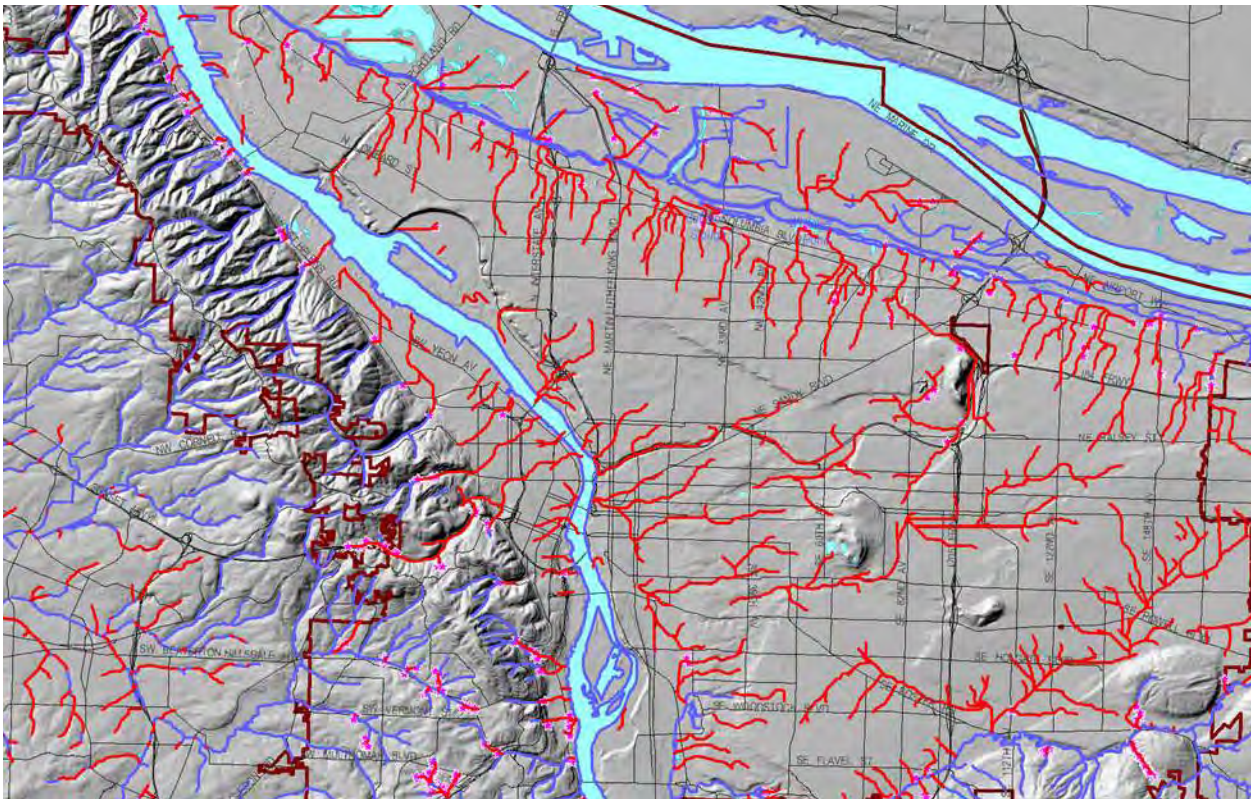
Figure 1-17: *There is only so much that the existing storm drain infrastructure can take. Green streets and parking lots can help relieve over-taxed systems.*

Inadequate Existing Stormwater Infrastructure

Urban development leads to an increase in impervious surfaces and a corresponding increase in surface runoff and pollutants from vehicles and other urban sources. The problem is exacerbated when increased stormwater runoff reaches a creek channel that is not capable of handling increased flows without significant erosion and degradation. Creeks with tributary areas having greater than 10% impervious surfaces are likely to have degraded water quality and habitat.

In San Mateo County, many of the regional storm drain systems were designed to outdated standards and lower service populations. Steady growth and urbanization over recent decades has left some local storm drain systems unable to handle the quantity of runoff produced by larger storm events, resulting in local flooding and associated damage. Besides being under-designed, other storm drain system inadequacies exist for a variety of reasons, including:

- Existing storm drain infrastructure has deteriorated
- Local neighborhood catch basins are inadequate
- Culverts have reduced capacity due to siltation
- Culverts are too low to drain by gravity during tidal conditions



SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-18: *The red lines indicate creeks and small streams in urban areas that have been replaced by decades of built underground pipe infrastructure. This scenario is all too common in communities throughout the United States.*

Flooding from an overwhelmed storm drain system results in a myriad of problems, such as:

- Storm drain backups and localized flooding
- Property damage
- Creek bank and bed erosion and downstream sedimentation
- Settled creek levees
- Restricted vehicular access
- Damaged roads
- Damaged or deteriorated bridge structural members

Water Pollution from Streets and Parking Lots

San Mateo County's storm drain system was designed to prevent local flooding by channeling stormwater runoff ultimately into the San Francisco Bay or the Pacific Ocean. This system provides no inherent water quality treatment. Stormwater runoff accounts for a majority of the pollutants entering local creeks and the San Francisco Bay. Potential pollutants include:

- Oil, grease, antifreeze, heavy metals from leaking and deteriorating cars and trucks, and brake pad and tire wear
- Pesticides, herbicides, and fertilizers from our residential and commercial landscapes
- Solvents and household chemicals (e.g., paint thinner, detergents, and paint)

• Animal waste, litter, decomposing vegetation, and sewage from leaks

• Construction debris, such as fresh concrete or mortar

Certain creeks, coastlines, and water bodies in San Mateo County have been identified under the Clean Water Act's section 303(d) as impaired by specific types of pollutants, such as sediment (see Appendix C). Sediment impairment of creeks is often caused by non-point sources associated with past and current land use practices. Conventional development practices may degrade the environment at a substantial cost to the larger community.

The Multiple Benefits of Using Green Streets and Parking Lots

Implementing landscape-based stormwater management facilities as part of green streets and parking lots in San Mateo County has the potential to minimize pollution, stream degradation, and localized flooding. Reintroducing bioretention into the hydrologic cycle reduces peak runoff rates and volumes by holding back and slowing down the water that would otherwise flow quickly into the storm drain system. By increasing natural storage and infiltration of rainwater, municipalities can slow peak flows and ease the burden of overwhelmed storm drain infrastructure. However, the benefits of using green streets and parking lots go beyond the obvious and include many ancillary environmental and community benefits.

Trash Removal

The effects of trash is another important water quality issue in San Mateo County. Improperly discarded trash is often washed into drainage systems during rains and finds its way into local creeks and the San Francisco Bay. In addition to physical pollution, trash can contribute chemical pollutants when it includes batteries, fluorescent tubes, and other such toxic waste. While there is no substitute for keeping trash out of the drainage system, green streets and parking lots can serve as localized collectors. Trash that would otherwise end up in San Mateo County's waterways can be regularly removed, recycled, or discarded in an environmentally appropriate way.

SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-19: *An example of the type of trash that often is conveyed into water bodies.*

SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-20: *Landscape-based stormwater facilities, such as this stormwater curb extension, can capture urban debris before it gets into creeks and other waterways.*

Community and Neighborhood Benefits

Green streets are not just about better stormwater management, but they are also about creating more beautiful and livable neighborhoods and communities in San Mateo County. Effectively "greening" the urban fabric helps provide a unique quality of life that increases the desirability of living in a particular community. Furthermore, incorporating green streets and parking lots offers people a very tangible way to learn about environmental sustainability. These types of projects can be built where we live, work, shop, learn, and play, and are constant reminders that rainwater is a resource, not a waste.

SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-21: *Neighborhood green streets are commonly seen as a community asset.*

Air Quality Benefits

Significant tree plantings throughout a parking lot or along a street site help mitigate local air quality issues. Trees help settle out particulate matter, reduce low-level ozone, and help mitigate the urban heat island effect. Light-colored permeable pavement further mitigates the heat island effect, since it increases the albedo, or diffuse reflectivity, of the paved area.

SOURCE: WWW.IA.NRCS.USDA.GOV

Figure 1-22: *Light-colored pervious paving within a parking lot helps reflect heat rather than absorb it.*

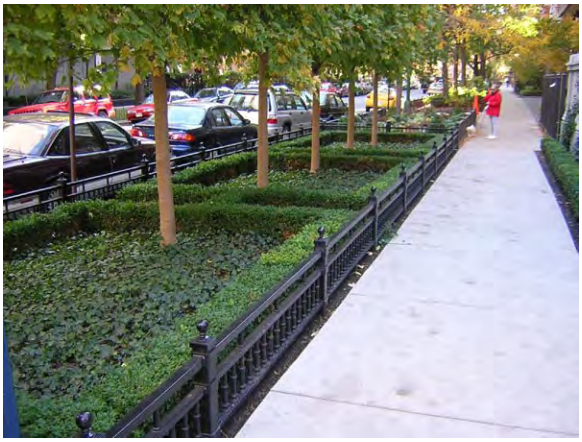
Economic Benefits

Providing more landscaping in the urban environment makes good economic sense. *Project Evergreen* (2008) states the following:

- *Smart Money* magazine indicated that consumers value a landscaped home up to 11.3% higher than its base price

- Studies by the University of Washington showed that drivers found it easier to locate businesses on a street when they were framed by trees and landscaping, rather than having this green material removed

- A recent study has also found that consumers are willing to pay, on average, a 12% premium for goods purchased in retail establishments that are accompanied by quality landscaping



SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-23: *This downtown residential street in Chicago, Illinois illustrates how ample landscaping can increase the appeal of a street.*

Potential Constraints for Green Street and Parking Lot Projects.

There are many constraints, both perceived and real, in implementing green street and parking lot projects. In an effort to fully understand the constraints specific to San Mateo County, a survey was developed and distributed to municipal public works staff in February 2008. This survey asked for direct input on potential constraints in four areas:

- Policy and Communication Constraints
- Site Characteristic Constraints
- Design-Related Constraints
- Construction/Long-Term Maintenance Constraints

The results of this survey is found in Appendix B of this guidebook. In addition, many of the "design-related" constraints are addressed in specific chapters of this guidebook.

Soil Conditions and Hydrology

Soil conditions and hydrology vary considerably in San Mateo County. In some cases, infiltration may be difficult due to steep hillsides or non-permeable soils (see Appendix C). However, there are ways to incorporate landscape-based stormwater facilities on streets and parking lots in steep conditions and poor infiltration soils. Chapter 5 discusses various methods for designing stormwater facilities in these difficult conditions.

Existing Impervious Area

Most of the impervious area in San Mateo County is concentrated in areas with flat topography (see Appendix C). An exception to this is with certain residential development areas that have occurred in moderately steep hillside conditions. Unfortunately, the areas of dense urban development also correspond with soil conditions that are typically unfavorable for infiltration. This represents a special challenge if a development project is subject to C.3 stormwater management requirements.

Current Stormwater Management Requirements

Current development or redevelopment projects that result in the addition or replacement of 10,000 square feet or more of impervious surface are subject to the C.3 requirements and are required to mitigate for water quality. In addition, all projects that create or replace one acre or more of impervious surface may be subject to flow and volume reduction requirements (see the C.3 Technical Guidance). Furthermore, there is a tremendous opportunity to significantly improve watershed health in the County by retrofitting existing streets and parking lots that are not slated for redevelopment and are not subject to C.3 stormwater requirements. There are many instances where stormwater facilities can be retrofitted in urban conditions that don't increase impervious area at all, but do increase landscape areas.In these conditions, the overall stormwater management goals are more flexible than what would be needed to comply with C.3 stormwater requirements for new and redevelopment projects.

SOURCE: NEVUE NGAN ASSOCIATES

Figure 1-24: *This green street retrofit project in Portland, Oregon was built not out of a requirement, rather it was built simply because it could be done.*

This chapter has described the principles of sustainable stormwater design, highlighted why better site-scale stormwater strategies are needed in San Mateo County, defined a green street and green parking lot, and described the unique conditions found in San Mateo County as they relate to stormwater management. The following provides a brief synopsis of what is included in the remaining chapters of the guidbook:

Chapter 2 introduces the stormwater design "toolbox" for San Mateo County. The toolbox contains a variety of site layout strategies and stormwater facility options that would be appropriate for San Mateo County. This chapter also highlights a three-step process for working through a site's stormwater design. The strategies found in Chapter 2 can be implemented in both new development and retrofit sites.

Chapter 3 describes the varying street and parking lot conditions found in San Mateo County and how these differing conditions affect the opportunities for green street and parking lot projects.

Chapter 4 illustrates how these different site strategies are applied to actual conditions within San Mateo County. Several example "before and after" sketches illustrate how sustainable stormwater design would look within the region.

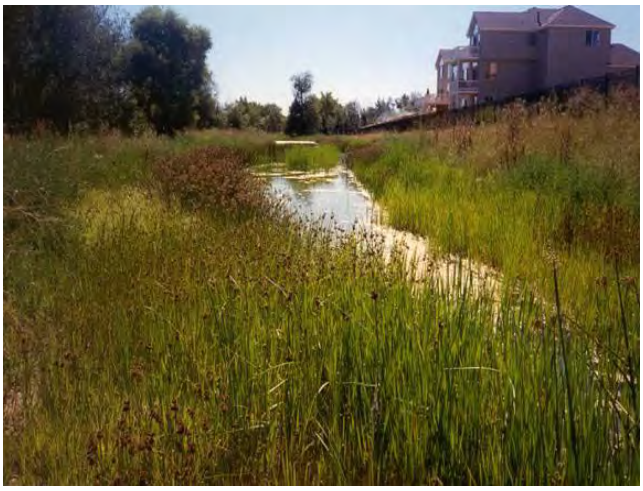
Chapter 5 of this guidebook offers more specific design details for green streets and provides practical advice on how to design and construct green street and parking lot projects.

Chapter 6 provides key considerations in implementing green street and parking lot projects/programs. This chapter also showcases the conceptual designs of several grant-funded green street and parking lot projects in San Mateo County. These projects will illustrate many of the strategies found within this guidebook and give the reader an opportunity to see real projects that will be built in the near future.

SOURCE: KEVIN ROBERT PERRY - CITY OF PORTLAND

Figure 1-25: *A residential green street example in Portland, Oregon.*

Urban Stormwater BMP Performance Monitoring



Prepared by
Geosyntec Consultants and
Wright Water Engineers, Inc.

Prepared under Support from
U.S. Environmental Protection Agency
Water Environment Research Foundation
Federal Highway Administration
Environmental and Water Resources Institute
of the American Society of Civil Engineers

October 2009

Disclaimer

This report was prepared by the organization(s) named below as an account of work sponsored by the Water Environment Research Foundation (WERF) and its partners on the International Stormwater Best Management Practices Database project. WERF's partners on the project include the U.S. Environmental Protection Agency (U.S. EPA), the Federal Highway Administration (FHWA), and the American Society of Civil Engineers – Environmental Water Resources Institute (ASCE-EWRI). Neither WERF, its partners on the project, members of WERF or ASCE-EWRI, the organization(s) named below, nor any person acting on their behalf: (a) makes any warranty, express or implied, with respect to the use of any information, apparatus, method, or process disclosed in this report or that such use may not infringe on privately owned rights; or (b) assumes any liabilities with respect to the use of, or for damages resulting from the use of, any information, apparatus, method, or process disclosed in this report.

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CITY OF SAN JOSE
CAPITAL OF SILICON VALLEY

Department of Public Works

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GUIDELINES FOR THE PLANNING, DESIGN AND CONSTRUCTION OF CITY STREETSCAPE PROJECTS

Updated June 2007

Photo on previous page:

MONTEREY ROAD (STATE ROUTE 82) MEDIAN ISLAND
LANDSCAPE IMPROVEMENT PROJECT
(Type 1 Enhanced)

Date of Project Completion: April 2004

Funding & Project Management provided by:
San José Redevelopment Agency

Capital Project Management & Inspection provided by:
City of San José, Department of Public Works

Design Consultant: Bellecci & Associates Inc.

Construction Contractor: Jos. J. Albanese, Inc.

If you have question regarding these guidelines, please call the Department of Public Works, Transportation and Development Services Division at (408) 535-3555.

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PART 1 - INTRODUCTION

These Guidelines describe the standard procedures and requirements for City Streetscape Projects during the Planning, Design and Construction Inspection stages. A City Streetscape Project ("Project") is a landscape improvement project, which is located within the street right-of-way *and* maintained by the City of San José's Department of Transportation ("DOT"). These Guidelines are intended for use by City and Redevelopment Agency Staff, Developers, Consultants, and Contractors. The City of San José Streetscape Projects addressed by these guidelines are either City funded Capital Improvement Projects that are designed and constructed by the City of San José's Department of Public Works ("DPW"), or private development Projects that are designed and constructed by a Developer. DPW provides the review of the improvement plans and construction inspection for both City funded Capital Improvement and private development Projects.

City Streetscape Projects include the following types of landscape improvements: median island landscaping, street tree plantings in parkstrips and tree wells, and the landscaping in front of soundwalls ("back-ups"). On June 9, 1994, the City Council approved a Median Island Landscape Program for median island development, which distinguished the type of landscaping required in median islands by the source of funding for maintenance: Type 1 - General Funds and Type 2 - Special District Funds. General definitions of Type 1, Type 1 Enhanced & Type 2 Projects are provided in Part 2 of these Guidelines. Refer to Appendix L for a copy of the City Council Approval Memo for the Median Island Landscape Program.

These Guidelines address the requirements of Landscape Improvement Plans for City Streetscape Projects that include detailed construction plans and specifications, which shall be hereafter referred to as "Project Landscape Improvement Plans". The Project Landscape Improvement Plans shall be designed by a Landscape Architect with a current State of California license ("Project Landscape Architect"). These Guidelines are to be used in conjunction with the DPW Standard Landscape Details and Specifications for City Streetscape Projects. Street improvements such as new median island curbing, sidewalks and street paving are outside the scope of these guidelines. Refer to the current DPW Standard Details and Specifications for the standards that apply to street improvement plans. The Project Landscape Architect and Contractor are required to comply with all relevant State regulations, City Municipal Codes and City Council Policies in the design and construction of City Streetscape Projects.

These Guidelines do not cover the Project requirements for planning permits and environmental clearances by the City's Department of Planning, Building and Code Enforcement. These Guidelines do not address the procedures for bid and award of construction contracts for City funded Capital Improvement Projects. Park-like landscape

improvements such as lakes, turf sport fields and play equipment, are not addressed in these Guidelines, even if these facilities are maintained by DOT with Special District Funding.

These Guidelines are not intended to supersede the guidelines or requirements of any other City Department that may have jurisdiction over the Project site (General Plans, Specific Plans, Public Art). Nor do these Guidelines supersede any requirements from outside Agencies such as Caltrans or the Santa Clara Valley Water District that may also have jurisdiction over the Project site. General Projects located downtown shall also comply with the current San José Redevelopment Agency ("SJRA") Greater Downtown Streetscape Master Plan.

- END OF PART 1 -

PART 2 - GENERAL DEFINITIONS OF TYPE 1, TYPE 1 ENHANCED & TYPE 2 CITY STREETSCAPE PROJECTS

2.1 **Type 1 City Streetscape Projects:** The maintenance for Type 1 Projects is paid for by the City's General Fund. Due to funding constraints, Type 1 Streetscapes are required to be low maintenance. High maintenance plantings of ground covers, perennials, turf and hedges are not allowed in Type 1 Projects. Ornamental structures such as water features and arbors are also not allowed in Type 1 Projects. An automatic, water-efficient irrigation system is required for all landscape improvements. The proposed design shall be coordinated with the existing streetscape. Refer to Appendix A for a detail of the standard features of Type 1 City Streetscape Projects. The following list describes the major elements required in standard Type 1 City Streetscape Projects.

2.1.1 **Type 1 Median Islands:**

- **Plant Materials:** Trees shall be planted the entire length of the planting areas in median islands, using the clearances described in Part 4.16.4 and Appendices A, D & E. The enhancement of median islands with shrubs and ground covers is allowed in Type 1 Projects only with advanced approval during the Planning Permit Stage; refer to Parts 2.2 and 4.1.
- **Decorative Median Paving**: All paving installed within median islands shall be decorative, with the City standard being concrete interlocking pavers such as Old Country Cobblestone Pavers manufactured by McNear or an approved equal. The use

CITY OF
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City of San José Recommended Temporary Traffic Control Plans

September 27, 2005

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Introduction

The Traffic Control Plans and associated text depicted in this Manual conforms with Part 6 of the Manual on Uniform Traffic Control Devices 2003 Edition (MUTCD) and the MUTCD 2003 California Supplement Chapter 6A – 6I.

The criteria of this Manual are intended to primarily apply to urban areas. Urban street traffic is typically characterized by relatively low speeds, wide ranges in traffic volume, narrower roadway lanes, frequent intersections, significant pedestrian traffic, bicyclists and frequent roadside obstacles.

This manual provides the basic standards for the safe movement of traffic upon highways and streets in accordance with Section 21400 of the California Vehicle Code. Traffic control includes safe protection for the public, motorist, cyclist, pedestrian and worker. It is the responsibility of the contractor or organization performing work on, or adjacent to, a roadway to install and maintain such devices which are necessary to provide safe passage for the traveling public through the work area and for the safety of the workers.

This text is intended to supplement the standards established by the Federal Department of Transportation and Caltrans by establishing construction area traffic control guidelines for work within the City of San Jose. The criteria for the position, location, manner of installation, and the use of such signs, lights and devices are furnished solely for the purpose of information and guidance.

No one set of signs or other traffic control devices can typically satisfy all conditions for a given project. At the same time, defining detailed standards that would be adequate to cover all applications is simply not practical. This Manual displays several diagrams that depict common applications of standard temporary traffic control devices and applications. The traffic control selected for each situation shall be based on street type, traffic conditions, duration of operation, physical constraints, and the nearness of the workspace to vehicle traffic, pedestrians, and bicyclists.

Envision
San José 2040

GENERAL PLAN

CITY OF SAN JOSE
CAPITAL OF SILICON VALLEY

Envision San José 2040

GENERAL PLAN

Building a City of Great Places

"We are blessed to live in this area with great beauty combined with a robust economy. We must plan carefully for the land remaining under our stewardship so that this good fortune is preserved and enhanced."

E.H. Renzel, Jr., San Jose Mayor 1945-1946
written in the month of his 100th birthday, August 2007

ENVISIONSAN JOSE2040

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available, resulting in maximum efficiency. The fiscal impact of potential land use and policy options will be given serious consideration and priority in the land use entitlement process. Goals, policies, and implementation actions throughout the General Plan address this important concept. The Plan incorporates policies from the City's Employment Lands Preservation Framework and several critical implementation policies to address the fiscal impacts of future land use decisions.

The *Envision San José 2040 General Plan* promotes a fiscally strong City, providing high-quality municipal services and acting as an advocate for regional policies that promote the strengths of our diverse and successful population. The Draft Plan incorporates policies that promote the City's fiscal health and which:

- Establish standards for the delivery of high-quality municipal services
- Carefully manage existing fiscal resources
- Encourage the cultivation of increased resources
- Focus new growth so as to minimize its fiscal impacts

Major Strategy #9 - Destination Downtown

Support continued growth in the Downtown as the City's cultural center and as a unique and important employment and residential neighborhood. Focusing growth within the Downtown will support the Plan's economic, fiscal, environmental, and urban design/placemaking goals.

Downtown San José is the cultural heart of San José and it provides employment, entertainment, and cultural activities more intensely than in any other area. The Downtown also consists of valuable historic resources, buildings with distinctive architecture, and unique neighborhoods where residents have convenient access to urban activities and amenities. As San José's largest and most vibrant urban area, Downtown contributes towards the positive identity of the City to the region, the nation and abroad.

Downtown San José is home to a growing number of companies whose employees enjoy the downtown life. Software and creative services businesses, in particular, offer technical skills and creative talent in San José's urban center. Silicon Valley's urban university, San José State, boasts award-winning capacity in product design, interactive media, and computer engineering. Between 2000 and 2010, the Downtown residential community grew by 8,000 people with the addition of new condominiums, lofts, and live-work units.

The Downtown includes the largest concentration of San José's civic and cultural facilities, including City Hall, the main library, the City's convention center, the arena, museums, theaters and outdoor gathering venues.

The Downtown is defined as:

- A vibrant urban center for living, working, entertaining
- The only location in the South Bay that actively promotes high-rise development

- A distinctive work environment for large and small companies
- Center to the City's arts, entertainment, culinary and sporting activities
- An eclectic mix of historic architecture side by side to award-winning contemporary urban design

The Plan recognizes the city's Downtown as the symbolic, economic, and cultural center of San José and supports a significant amount of job and housing growth within the Downtown area. The Draft Plan policies address how the Downtown is a:

- Unique urban destination
- Cultural center of the Silicon Valley
- Growing employment and residential center

Major Strategy #10 - Life Amidst Abundant Natural Resources

Promote access to the natural environment and a favorable climate as important strengths for San José by building a world-class trail network, reinforcing the Greenline / Urban Growth Boundary as the limit of the City's urbanized area and to preserve the surrounding hillsides largely as open space, and by adding parks and other recreational amenities to serve existing and new populations.

The Greenline/Urban Growth Boundary defines the ultimate perimeter of urbanization in San José. Besides setting limits to urban development as a growth management and fiscal sustainability strategy, the Greenline/Urban Growth Boundary is intended to develop a clearer identity for San José by defining where urban development ends and by establishing policies to preserve valuable open space resources. Natural resources surrounding the lands within the Greenline/Urban Growth Boundary are the inspiration for this concept.

The Greenline/Urban Growth Boundary demonstrates the strong, long-standing commitment of both the City of San José and the County of Santa Clara that urban development should occur only within the Urban Service Areas of cities where it can safely and reasonably be accommodated and where urban services can efficiently be provided. Lands outside of the Greenline/Urban Growth Boundary are identified as those that are intended to remain permanently rural in character and that should remain under the jurisdiction of the County. Both the City and the County are committed to the success of this arrangement and will continue to develop and implement consistent land use plans and development policies for lands of mutual concern both within and outside of the Greenline/Urban Growth Boundary. This commitment is reflected in the Greenline/Urban Growth Boundary goals and policies of both General Plans.

Major Strategy #11 - Design for a Healthful Community

Support the physical health of community members by promoting walking and bicycling as commute and recreational options, encouraging access to healthful foods, and supporting the provision of health care and safety services.

The "Design for a Healthful Community" concept embodies a very broad range of topics, and "Design for a Healthful Community" strategies are integrated throughout the General Plan to establish a policy framework to shape and grow a city that provides for the physical health of its residents. General Plan policies support good nutrition and healthful air and water, protect the community from human-made and natural hazards and disasters, provide for economic opportunities that meet the needs of all residents, and provide for the equitable distribution of public resources, including public health facilities, throughout the City.

To further the Healthful City concept, the Land Use/Transportation Diagram, and the goals and policies related to Quality Neighborhoods, Urban Villages, Urban Design, Complete Streets, and Transportation, encourage physical activity by creating "complete" communities where most individuals' daily needs can be met walking or biking on safe and convenient paths and routes. Parks, Trails, Open Space, and Recreation policies also encourage activity by promoting good and convenient access to a large and diverse variety of parks, trails and recreations facilities for all City residents.

As a key factor to encourage the health of its residents, the Land Use/Transportation Diagram, and the Quality Neighborhoods and Land Use policies address improving access to healthful foods, particularly fresh produce. To this end, the General Plan also supports the development of urban agriculture, and the preservation of the existing agricultural lands adjacent to San José to increase the supply of locally-grown, healthful foods. The General Plan further supports Healthful Community regulatory land use policies, enabling the operation of farmer's markets, urban farming activities, and promoting the availability of healthful foods while limiting access to alcohol at retail locations.

The *Envision San José 2040 General Plan* Land Use / Transportation Diagram and policies promote the physical health of San José's community by:

- Promoting access to healthful foods
- Encouraging an active lifestyle
- Supporting health care and safety services

Major Strategy #12 - Plan Horizons and Periodic Major Review

Ensure that the Plan addresses the current community context and values and closely monitor the achievement of key Plan goals through a periodic major review of the General Plan and the use of Plan Horizons to phase implementation of the Plan over time.

The Plan provides a tool for phasing the development of new Urban Village areas and gives highest priority to the location of new housing growth in the Downtown, connecting transit corridors, BART station area, and North San José. The *Envision General Plan* establishes a 4-year Major Review cycle, which provides an opportunity for a community stakeholder task force and the City Council to evaluate significant changes in the planning context and the City's achievement of:

- Planned job and J/ER goals
- Implementation of the Urban Village concept
- Environmental indicators, including greenhouse gas reduction and the Green Vision
- Affordable housing needs

The Phasing Plan policies also include flexibility to allow the implementation of Urban Villages to be responsive to market conditions, while meeting overall Plan objectives.

Growth Areas

The Land Use / Transportation Diagram, General Plan policies and the Growth Areas concept diagram identify specific areas of San José which are planned to accommodate the majority of the City's job and housing growth. The planned location of job and housing growth capacity supports the City's long-term goal to emphasize growth within the Downtown, North San José and Specific Plan areas, while focusing new job and housing growth capacity in identified transit corridor, transit station area, commercial center and neighborhood Urban Village Growth Areas. The specific amounts of job and housing growth capacity for each of the Growth Areas are indicated in Appendix 5 – Growth Areas Planned Capacity by Horizon.

Regional Transit and Transit-Oriented Urban Villages include vacant or under-utilized lands within close proximity of an existing or planned light rail, BART, Caltrain or Bus Rapid Transit (BRT) facility. Commercial Center Urban Villages include vacant or under-utilized lands in existing, large-scale commercial areas (e.g., Oakridge Mall, Winchester Boulevard, Bascom Avenue, etc.) Neighborhood Urban Villages are smaller neighborhood-oriented commercial sites with redevelopment potential. While the Neighborhood Urban Villages are not located in proximity to major transit facilities, their intensification could serve to create a vibrant village setting within easy access of the nearby neighborhood. For all of the Urban Village areas it is expected that the existing amount of commercial square footage would be retained and enhanced as part of any redevelopment project so that existing commercial uses within San José are never diminished.

The following text summarizes the special characteristics of each one of the City's Growth Areas, with the Growth Areas Diagram following the text:

- Downtown
- Specific Plans
- North San José
- Employment Lands
- Regional Transit Stations
- San José Transit Urban Villages
- Commercial Center Urban Villages
- Neighborhood Urban Villages

Downtown

The *Envision San José 2040 General Plan* reinforces the importance of San José's Downtown as the physical and symbolic center of the City. Planned growth capacity and the General Plan policies are intended to further support the growth and maturation of the Downtown as a great place to live, work or visit. The *Envision San José 2040 General Plan* maintains and augments the City's *Downtown Strategy 2000* to support regional transit use, continue the development of the Downtown as a regional job center and to support continued development of high-rise development within the Downtown area.

The boundaries for the Downtown Growth Area (also referred to as the "Downtown Area" and historically as the "Downtown Core") are defined on the Planned Growth Areas Diagram. The Downtown Growth Area is bounded by Julian Street, North 4th Street, East St. John Street, 7th Street, East San Fernando Street, South 4th Street, Interstate 280, the Union Pacific Railroad line, Stockton Avenue, Taylor Street, and Coleman Avenue.

Ambitious job and housing growth capacity is planned for the Downtown and supported by Downtown Strategy 2000. This growth capacity is important to achieve multiple City goals, including support for regional transit systems. It also helps to advance all elements of the General Plan Vision.

Specific Plans

Specific Plan areas have played a central role in the City's ongoing commitment to providing new housing through transit-oriented development projects. While a few of the Specific Plans have been fully implemented, several continue to provide important growth capacity. The City's adopted Specific Plans generally have a residential orientation, providing significant capacity for residential and mixed-use development at important infill sites throughout the City and often in proximity to the Downtown. The *Envision General Plan* maintains the existing growth capacity and residential focus of the Specific Plan areas, with the exception of the Alviso Master Plan, which has an employment focus and expanded job growth capacity provided through the *Envision General Plan*.

The Alviso Master Plan preserves the existing Alviso Village area and supports significant employment growth as an extension of the City's key North San José employment district. Within the Alviso Plan area, the Water Pollution Control Plant lands have been identified as a significant opportunity for new employment land areas, and in particular to provide an opportunity for new light industry or manufacturing activity jobs. According to current occupancy data and the Envision *Job Growth Projections and Employment Land Demand* report, there is a significant need for additional industrial land of this type. Because the other Specific Plan areas are generally built-out and/or located in areas with a lesser degree of transit access, employment growth in those areas is more focused on commercial or industrial uses that support local residences and businesses.

The *Envision General Plan* provides a limited amount of new residential and job growth capacity in other Specific Plan areas, consistent with the overall Plan goal of focusing new growth in proximity to transit and other City services. Because the Specific Plans were developed through extensive community-based planning processes, the *Envision General Plan* incorporates, with only very limited modification, the land uses designated within the Specific Plan areas. Further review and updating of the Specific Plans in the future will be important to fully realize the goals and policies of the *Envision General Plan*.

North San José

North San José is the City's largest employment district, home to many important leading technology companies and a key growth area for the City. The *Envision General Plan* augments the North San José Area Development Policy, providing growth capacity for up

to 97,000 new jobs and 32,000 new housing units to further its development as a regional employment center.

The City's North San José Area Development Policy continues to be the key implementation document for this area. The Policy includes a phasing plan and a Traffic Impact Fee which link job and housing growth, and requires some amount of both in order to fund transportation improvements. This Plan considers the new residential neighborhoods addressed in the North San José Area Development Policy, North San José Urban Design Guidelines and North San José Neighborhoods Plan as already planned Urban Villages. Those completed policy documents will serve as an Urban Village Plan for each neighborhood area.

Employment Lands

Significant job growth is planned through intensification of each of the City's Employment Land areas, including the Monterey Corridor, Edenvale, Berryessa/International Business Park, Mabury, East Gish and Senter Road, and North Coyote Valley as well as North San José. These Employment Lands are planned to accommodate a wide variety of industry types and development forms, including high-rise and mid-rise office or research and development uses, heavy and light industrial uses and supporting commercial uses to respond to the projected demand for each type of industrial land. Three areas are designated as Employment Centers because of their proximity to regional transportation infrastructure. These include the North San José Core Area along North First Street, the portion of the Berryessa/International Business Park in close proximity to the planned Milpitas BART station and existing Capitol Avenue Light Rail stations, and the Old Edenvale area, which because of its access to light rail, is also planned for additional job growth. The *Envision General Plan* does not support conversion of industrial lands to residential use, nor does it include housing growth capacity for these areas.

Regional Transit Stations

To support the City's growth as a regional job center and to encourage greater utilization of regional transit infrastructure, the General Plan provides significant new job growth capacity within Regional Transit (BART, Caltrain and High-Speed Rail) Urban Villages. Both the Lundy/Milpitas and Berryessa BART station areas support large amounts of new mid-rise and high-rise employment uses, while the Berryessa BART Urban Village is also planned for additional housing development. The Alum Rock BART station area accommodates both job and housing growth, on a more limited scale, given the local site characteristics and neighborhood interface.

San José Transit Urban Villages

A large and balanced amount of job and housing growth capacity is planned for the Transit Villages and Corridors. The goal is to maximize the opportunity for creating new mixed-use villages in these areas. While the BART area job capacity is planned primarily for mid-rise and high-rise offices, the Light Rail Villages should provide more opportunity for retail and service jobs that benefit from close proximity to residential use. While the BART system serves as a regional transit line, bringing workers from throughout the region to employment centers within San José, the light rail system is more appropriate for shorter com-

mute trips and is also less likely to generate land use compatibility concerns. Accordingly, it is appropriate to include more residential and retail growth capacity along the light rail system. The Oakridge Mall Light Rail station area is particularly of interest because of its size and high level of unrealized potential to support a walkable, mixed-use community, and also the amount and diversity of established commercial uses and other services already located within the area. The boundaries for the Urban Village Growth Areas are designated on the Planned Growth Areas Diagram and on the Land Use/Transportation Diagram.

Commercial Center Urban Villages

While the Commercial Centers are less directly connected to transit than other Growth Areas, they contain large parcels which may have greater potential for redevelopment and are generally located in areas with a high degree of accessibility which is advantageous for intensified commercial development. Providing residential growth capacity in the Commercial Center Growth Areas is a potential catalyst to spur the redevelopment and enhancement of existing commercial uses while also transforming them into mixed-use Urban Village type environments. At the same time, their typically more suburban settings may create some challenge to such revitalization. The Commercial Centers with closer proximity to other growth areas and transit (e.g., North Bascom Avenue) or in proximity to established, more intense uses (e.g., De Anza Blvd.) may have greater near-term potential for transformation into Urban Village settings. A modest and balanced amount of new housing and job growth capacity is planned for the Commercial Centers in order to support their intensification as both employment and housing centers, support potential expansion of existing retail activity, and add a mix of employment uses while also recognizing that transit-oriented sites should be given more priority for accommodating new growth. The boundaries for the Urban Village Growth Areas are designated on the Planned Growth Areas Diagram and on the Land Use/Transportation Diagram.

Neighborhood Urban Villages

To support the General Plan goal of providing broad access to mixed-use Urban Villages for all areas of the City, the Plan establishes Neighborhood Urban Villages as one of the identified Growth Areas. In keeping with the *Envision General Plan's* goal to support job growth, Neighborhood Urban Villages are planned to accommodate job growth along with a small amount of new housing. Job growth within Urban Village Areas is planned to focus on neighborhood-serving office, retail, and other commercial uses while providing opportunities for a wide variety of employment activity. Because implementation of the Neighborhood Urban Villages will require redevelopment of existing commercial sites which serve the surrounding community, the Plan anticipates retention of all existing commercial activities within the Urban Village area. Modest increases in housing and employment within the Neighborhood Urban Villages should also support expansion of retail activity. It may be difficult to attract large numbers of new jobs to these locations given their relatively small scale and separation from other employment areas, but some additional job growth should be possible to support surrounding uses. The boundaries for the Urban Village Growth Areas are designated on the Planned Growth Areas Diagram and on the Land Use/Transportation Diagram.

Scientific Investigations Report 2007–5156

Evaluation of Street Sweeping as a Stormwater-Quality-Management Tool in Three Residential Basins in Madison, Wisconsin

By William R. Selbig and Roger T. Bannerman

Return to main report.

Appendixes

Appendix 2. Quality assurance and quality control

Appendix 2-1. Results of blank-sample analyses.

Appendix 2-2. Results of replicate-sample analyses.

Appendix 2. Quality assurance and quality control

Field and sample-processing equipment blanks were collected at the control and test-basin monitoring stations to evaluate the integrity of the stormwater-quality sampling process, identify if sample contamination existed and if so, to identify possible sources. Blank samples were obtained by drawing deionized water through the suction line and sampler into a collection bottle. The Teflon sample line and automatic sampler were not cleaned before obtaining blank samples. Blank water collected in the 10-liter glass sample bottle was then split using a Teflon-lined churn splitter into plastic laboratory-prepared sample bottles. Samples were placed on ice and delivered to the Madison Department of Public Health (MDPH) for analysis. Deionized blank water was also used to isolate individual elements of the sampling process from source to delivery. These samples were not delivered to the MDPH unless erroneous concentrations were found in the original blank sample. Blank-sample results are detailed in table 2-1. Replicate samples were also collected to evaluate the inherent variability in the sampling analyses and methods. The bias and variability identified by analysis of blanks and replicates were within acceptable limits except for dissolved zinc, dissolved chloride, and total cadmium.

Total zinc concentrations often were above detectable limits but were typically an order of magnitude lower than those measured in stormwater-quality samples. Dissolved zinc concentrations, however, were detected in blanks at concentrations greater than in field samples

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JULY 1992

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United States Environmental Protection Agency
Office of Water (4203) Washington, DC 20460
EPA-841-B-00-005 October 2000

LOW-IMPACT DEVELOPMENT CENTER

Low Impact Development (LID)

A Literature Review

EXECUTIVE SUMMARY

A literature review was conducted to determine the availability and reliability of data to assess the effectiveness of low impact development (LID) practices for controlling stormwater runoff volume and reducing pollutant loadings to receiving waters.

Background information concerning the uses, ownership and associated costs for LID measures was also compiled. In general LID measures are more cost effective and lower in maintenance than conventional, structural stormwater controls. Not all sites are suitable for LID. Considerations such as soil permeablility, depth of water table and slope must be considered, in addition to other factors. Further, the use of LID may not completely replace the need for conventional stormwater controls.

Maintenance issues can be more complicated than for conventional stormwater controls because the LID measures reside on private property. In most instances, homeowners agree to only the first year of maintenance. Homeowner associations could be a mechanism for providing long-term maintenance to these areas. Generally, bioretention facilities require replacement of dead or diseased vegetation, remulching as needed, and replacement of soils after 5–10 years. Grass swales require periodic mowing and removal of sediments. Maintenance of permeable pavements requires annual high-powered vacuuming of the area to remove sediments.

Several studies have been conducted to analyze the effectiveness of various LID practices based on hydrology and pollutant removal capabilities. Bioretention areas, grass swales, permeable pavements and vegetated roof tops were the most common practices studied. These techniques reduce the amount of Effective Impervious Area (EIA) in a watershed. EIA is the directly connected impervious area to the storm drain system and contributes to increased watershed volumes and runoff rates. There are documented case studies that conclusively link urbanization and increased watershed imperviousness to hydrologic impacts on streams. Existing reports and case studies provide strong evidence that urbanization negatively affects streams and results in water quality problems such as loss of habitat, increased temperatures, sedimentation and loss of fish populations (USEPA, 1997)

In general bioretention areas were found to be effective in reducing runoff volume and in treating the first flush (first ½ inch) of stormwater. Results from three different studies indicate that removal efficiencies were quite good for both metals and nutrients. Removal rates for metals were more consistent than for nutrients. Removal rates for metals ranged from 70–97% for lead, 43–97% for copper and 64–98% for zinc. Nutrient removal was more variable and ranged from 0–87% for phosphorus, 37–80% for Total Kjeldahl Nitrogen, <0–92% for ammonium and for nitrate <0–26%. Effluent volumes were lower than influent volumes. These studies were conducted by means of simulated rainfall events. Analysis of actual long-term rainfall events would produce more reliable data.

The effectiveness of grass swales was also quite good for both pollutant removal and runoff volume reduction. A study of three different sites in the United States reveal similar results despite the differences in location. In general, performance of swales is

dependant on not only channel length, but also longitudinal slope and the use of check dams to slow flows and allow for greater infiltration. Further, the removal of metals was found to be directly related to the removal rate of total suspended solids, and the removal rate of metals was greater than removal of nutrients.

Reduction of impervious surfaces can greatly reduce the volume of runoff generated by rainfall. Several methods can be employed to reduce total impervious surface area. Permeable pavements and vegetated rooftops are two methods to accomplish this goal. Vegetated rooftops have been used extensively in Germany for more than 25 years and results show up to 50% reduction in annual runoff in temperate climates. Many opportunities exist to retrofit these systems into older highly urbanized areas of the United States. The Philadelphia project case study provides an example of this practice.

Permeable pavements can also reduce impervious surfaces. However, they are more expensive to construct than traditional asphalt pavements. Costs of these systems may be off set by the reduction of traditional curb and gutter systems to convey stormwater. Benefits of these alternate pavement types include better infiltration, ground water recharge, reduction in runoff volume and treatment of stormwater for pollutants. The study conducted in Tampa, Florida outlines these benefits as well as the opportunity to retrofit permeable pavements into existing parking lots with little or no loss of parking space. Less than 20% of rainfall was converted to runoff when using permeable pavements. Study results from the University of Washington, compare several different treatments of varying permeablility. The study shows that the higher the amount of perviousness of the treatment, the greater the reduction of runoff volume and pollutant loadings.

The use of LID is relatively new and not widespread. Most of the available data are from Prince George's County, Maryland, which pioneered the use of LID. The data available for bioretention analysis were from single simulated storm events in actual bioretention facilities or from laboratory constructed and tested bioretention systems. The data for grass swales were for only a few storm events, collected over a short period of time. The only available data for a long-term study came from the Aquarium parking lot in Tampa, Florida and the Washington permeable pavement project. More long-term analysis is required to more accurately assess the effectiveness of LID and to determine long term trends.

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1 LOW IMPACT DEVELOPMENT

1.1 Introduction

Low impact development (LID) is a relatively new concept in stormwater management. LID techniques were pioneered by Prince George's County, Maryland, in the early 1990's, and several projects have been implemented within the state. Some LID principles are now being applied in other parts of the country, however, the use of LID is infrequent and opportunities are often not investigated. The purpose of this report is to conduct a literature review to determine existing information about the application of LID in new development and existing urbanized areas, including ownership, operation and maintenance issues. A related objective was to locate relevant studies of LID projects, which would provide evidence of the effectiveness of LID in retaining predevelopment hydrology and as a mechanism for pollutant removal for stormwater. The data from the studies were analyzed for usefulness and validity and the findings are summarized.

LID is a site design strategy with a goal of maintaining or replicating the pre-development hydrologic regime through the use of design techniques to create a functionally equivalent hydrologic landscape. Hydrologic functions of storage, infiltration, and ground water recharge, as well as the volume and frequency of discharges are maintained through the use of integrated and distributed micro-scale stormwater retention and detention areas, reduction of impervious surfaces, and the lengthening of flow paths and runoff time (Coffman, 2000). Other strategies include the preservation/protection of environmentally sensitive site features such as riparian buffers, wetlands, steep slopes, valuable (mature) trees, flood plains, woodlands and highly permeable soils.

LID principles are based on controlling stormwater at the source by the use of micro-scale controls that are distributed throughout the site. This is unlike conventional approaches that typically convey and manage runoff in large facilities located at the base of drainage areas. These multifunctional site designs incorporate alternative stormwater management practices such as functional landscape that act as stormwater facilities, flatter grades, depression storage and open drainage swales. This system of controls can reduce or eliminate the need for a centralized best management practice (BMP) facility for the control of stormwater runoff. Although traditional stormwater control measures have been documented to effectively remove pollutants, the natural hydrology is still negatively affected (inadequate base flow, thermal fluxes or flashy hydrology), which can have detrimental effects on ecosystems, even when water quality is not compromised (Coffman, 2000). LID practices offer an additional benefit in that they can be integrated into the infrastructure and are more cost effective and aesthetically pleasing than traditional, structural stormwater conveyance systems.

Conventional stormwater conveyance systems are designed to collect, convey and discharge runoff as efficiently as possible. The intent is to create a highly efficient drainage system, which will prevent on lot flooding, promote good drainage and quickly convey runoff to a BMP or stream. This runoff control system decreases groundwater

recharge, increases runoff volume and changes the timing, frequency and rate of discharge. These changes can cause flooding, water quality degradation, stream erosion and the need to construct end of pipe BMPs. Discharge rates using traditional BMPs may be set only to match the predevelopment peak rate for a specific design year. This approach only controls the rate of runoff allowing significant increases in runoff volume, frequency and duration of runoff from the predevelopment conditions and provides the mechanisms for further degradation of receiving waters (Figure 1).

LID has often been compared to other innovative practices, such as Conservation Design, which uses similar approaches in reducing the impacts of development, such as reduction of impervious surfaces and conservation of natural features. Although the goals of Conservation Design protect natural flow paths and existing vegetative features, stormwater is not treated directly at the source. Conservation Design protects large areas adjacent to the development site and stormwater is directed to these common areas.

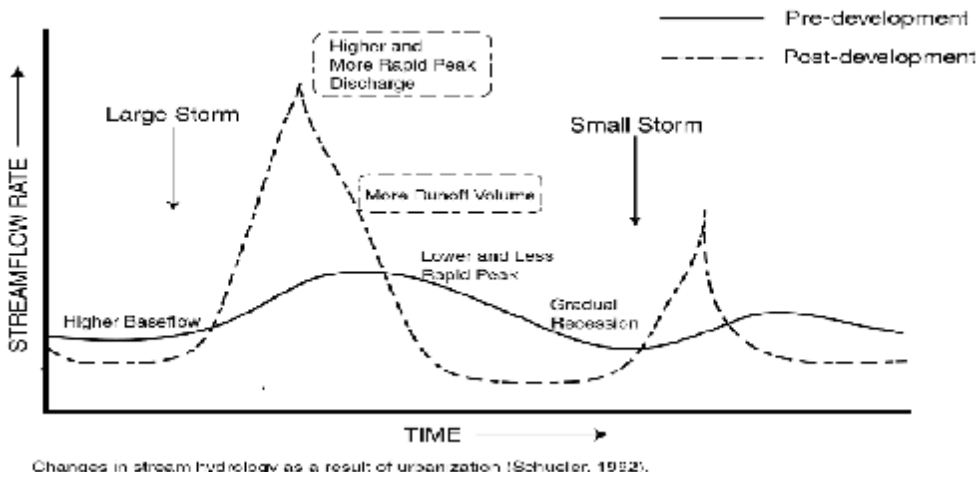

Figure 1: Changes in Stormwater Hydrology as a Result of Urbanization

Although this approach protects trees and does reduce runoff, there is still potentially a significant amount of connected impervious area and centralized stormwater facilities that may contribute to stream degradation through stormwater volume, frequency and thermal impacts. Therefore, the hydrologic and hydraulic impacts of this approach on receiving waters may still be significant, although the volume and flows will be less than without the conservation design. The stormwater control measures used in Conservation Design are off-site and therefore not the individual property owner's responsibility. However, maintenance is generally provided by the homeowners association and financed through association fees.

1.2 Benefits and Limitations

The use of LID practices offers both economical and environmental benefits. LID measures result in less disturbance of the development area, conservation of natural features and can be less cost intensive than traditional stormwater control mechanisms. Cost savings for control mechanisms are not only for construction, but also for long-term

maintenance and life cycle cost considerations. For example, an alternative LID stormwater control design for a new 270 unit apartment complex in Aberdeen, NC will save the developer approximately 72% or $175,000 of the stormwater construction costs. On this project, almost all of the subsurface collection systems associated with curb and gutter projects have been eliminated. Strategically located bioretention areas, compact weir outfalls, depressions, grass channels, wetland swales and specially designed storm water basins are some of the LID techniques used. These design features allow for longer flow paths, reduce the amount of polluted runoff and filter pollutants from stormwater runoff (Blue Land, Water and Infrastructure, 2000).

Today many states are facing the issue of urban sprawl, a form of development that consumes green space, promotes auto dependency and widens urban fringes, which puts pressure on environmentally sensitive areas. "Smart growth" strategies are designed to reconfigure development in a more eco-efficient and community oriented style. LID addresses many of the environmental practices that are essential to smart growth strategies including the conservation of open green space. LID does not address the subject of availability of public transportation.

LID provides many opportunities to retrofit existing highly urbanized areas with pollution controls, as well as address environmental issues in newly developed areas. LID techniques such as rooftop retention, permeable pavements, bioretention and disconnecting rooftop rain gutter spouts are valuable tools that can be used in urban areas. For example, stormwater flows can easily be directed into rain barrels, cisterns or across vegetated areas in high-density urban areas. Further, opportunities exist to implement bioretention systems in parking lots with little or no reduction in parking space. The use of vegetated rooftops and permeable pavements are 2 ways to reduce impervious surfaces in highly urbanized areas.

LID techniques can be applied to a range of lot sizes. The use of LID, however, may necessitate the use of structural BMPs in conjunction with LID techniques in order to achieve watershed objectives. The appropriateness of LID practices is dependent on site conditions, and is not based strictly on spatial limitations. Evaluation of soil permeability, slope and water table depth must be considered in order to effectively use LID practices. Another obstacle is that many communities have development rules that may restrict innovative practices that would reduce impervious cover. These "rules" refer to a mix of subdivision codes, zoning regulations, parking and street standards and other local ordinances that determine how development happens (Center for Watershed Protection, 1998). These rules are responsible for wide streets, expansive parking lots and large-lot subdivisions that reduce open space and natural features. These obstacles are often difficult to overcome.

Additionally, community perception of LID may prevent its implementation. Many homeowners want large-lots and wide streets and view reduction of these features as undesirable and even unsafe. Furthermore, many people believe that without conventional controls, such as curbs and gutters and end of pipe BMPs, they will be required to contend with basement flooding and subsurface structural damage.

2 LOW IMPACT DEVELOPMENT PRACTICES

LID measures provide a means to address both pollutant removal and the protection of predevelopment hydrological functions. Some basic LID principles include conservation of natural features, minimization of impervious surfaces, hydraulic disconnects, disbursement of runoff and phytoremediation. LID practices such as bioretention facilities or rain gardens, grass swales and channels, vegetated rooftops, rain barrels, cisterns, vegetated filter strips and permeable pavements perform both runoff volume reduction and pollutant filtering functions.

2.1 Bioretention

Bioretention systems are designed based on soil types, site conditions and land uses. A bioretention area can be composed of a mix of functional components, each performing different functions in the removal of pollutants and attenuation of stormwater runoff (Figure 2).

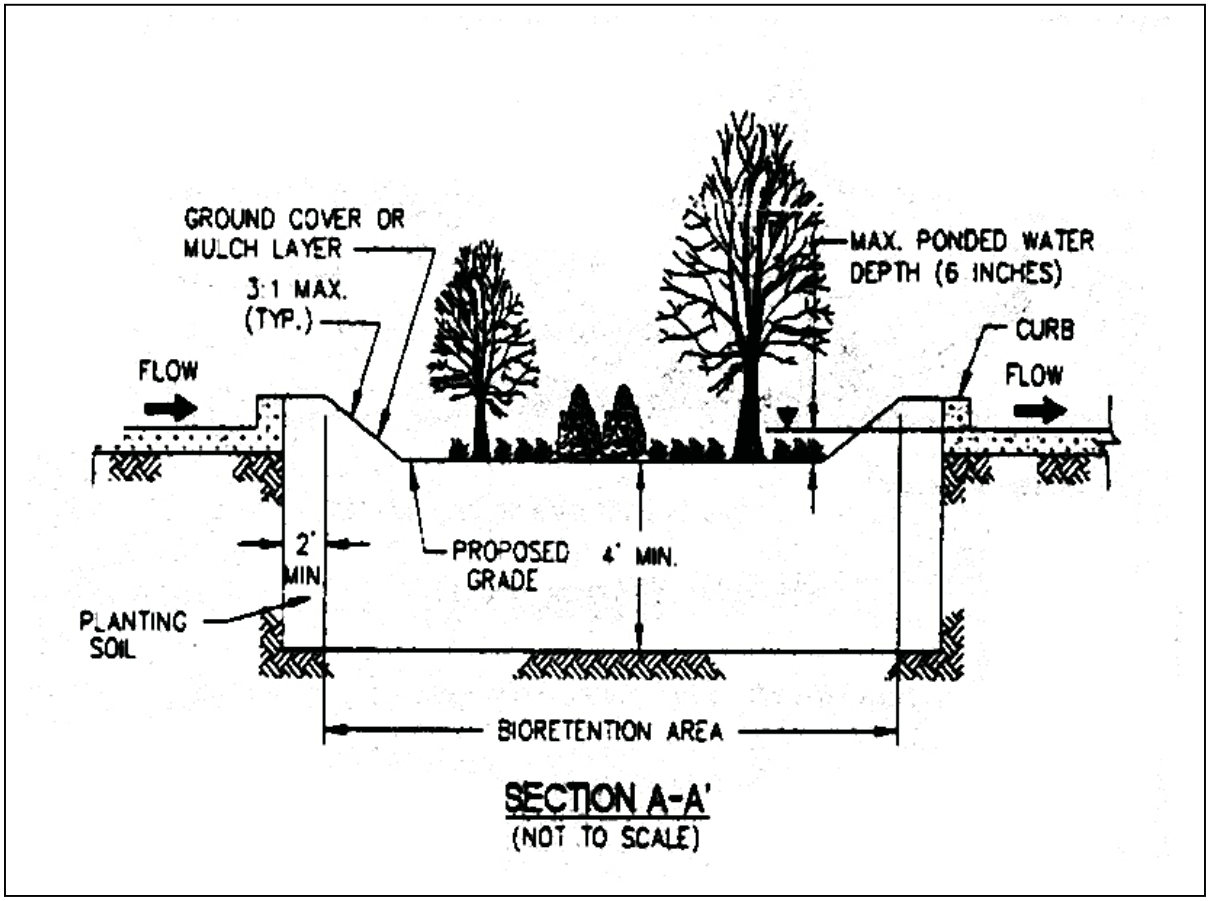

Figure 2: Typical Bioretention System (Prince George's County Department of Environmental Resources, 1993)

Six typical components found in bioretention cells:

- Grass buffer strips reduce runoff velocity and filter particulate matter.
- Sand bed provides aeration and drainage of the planting soil and assists in the flushing of pollutants from soil materials.
- Ponding area provides storage of excess runoff and facilitates the settling of particulates and evaporation of excess water.
- Organic layer performs the function of decomposition of organic material by providing a medium for biological growth (such as microorganisms) to degrade petroleum-based pollutants. It also filters pollutants and prevents soil erosion.
- Planting soil provides the area for stormwater storage and nutrient uptake by plants. The planting soils contain some clays which adsorb pollutants such as hydrocarbons, heavy metals and nutrients.
- Vegetation (plants) functions in the removal of water through evapotranspiration and pollutant removal through nutrient cycling.

Bioretention facilities are less cost intensive than traditional structural stormwater conveyance systems. Construction of a typical bioretention area in Prince George's County, Maryland is between $5,000 and $10,000 per acre drained, depending on soil type (Weinstein, 2000). Other sources estimate the costs for developing bioretention sites at between $3 and $15 per square foot of bioretention area. Design guidelines recommend that bioretention systems occupy 5-7% of the drainage basin. Additional savings can be realized in reduced construction costs for storm drainpipe. For example, bioretention practices reduced the amount of storm drain pipe at a Medical Office building in Prince George's County, Maryland from 800 to 230 feet, which resulted in a cost savings of $24,000 or 50% of the overall drainage cost for the site (Dept. of Env. Resources, 1993).

Components of the bioretention area should meet required guidelines in order to provide the most productive system possible. The mulch layer should be approximately 2-3 inches thick and replaced annually. Soil should be tested for several criteria before being used.

- pH range 5.5–6.5
- Organic matter 1.5–3.0%
- Magnesium (Mg) 35lbs/acre
- Phosphorus (P2O5) 100lbs/acre
- Potassium (K2O) 85lbs/acre
- Soluble salts < 500 ppm

Plant material should be obtained from certified nurseries that have been inspected by state or federal agencies (Dept. of Env. Resources, 1993). Native species should be used and selected according to their moisture regime, morphology, susceptibility to pests and diseases and tolerance to pollutants. Selection of plant species should be based on site conditions and ecological factors. A minimum of three species of trees and three species of shrubs should be selected to insure diversity, differing rates of transpiration and ensure a more constant rate of evapotransportation and nutrient and pollutant uptake throughout the growing season (Dept. of Env. Resources, 1993). Species that require regular maintenance should be avoided or restricted. Prince George's County recommends a warranty be established with the nursery as part of the plant installation, and should include care and 80% replacement of plants for the first year.

Table 1: Example Maintenance Schedule for Bioretention Areas (Prince George's County, Department of Environmental Resources, 1993)

Description	Method	Frequency	Time of Year
Soil			
Inspect and Repair Erosion	Visual	Monthly	Monthly
Organic Layer			
Remulch void areas	By Hand	As Needed	As Needed
Remove previous mulch layer before applying new layer (optional)	By Hand	Once a Year	Spring
Additional mulch added (optional)	By Hand	Once a Year	Spring
Plants			
Remove and replace all dead and diseased vegetation that cannot be treated	See Planting Specifications	Twice a Year	Mar 15–Apr 30 and Oct 1–Nov 30
Treat all diseased trees and shrubs	Mechanical or by Hand	N/A	Varies, depends on insect or disease infestation
Water of plant materials, at the end of the day for 14 consecutive days after planting	By Hand	Immediately after Completion of Projects	N/A
Replace stakes after one year	By Hand	Once a Year	Remove only in the Spring
Replace deficient stakes or wires	By Hand	N/A	As Needed

Annual maintenance is required for the overall success of bioretention systems. This includes maintenance of plant material, soil layer and the mulch layer. A maintenance schedule outlining methods, frequency and time of year for bioretention maintenance should be developed. Table 1 is a typical maintenance checklist. Plants will provide enhanced environmental benefit over time as root systems and leaf canopies increase in size and pollutant uptake and removal efficiencies. Soils, however, begin filtering pollutants immediately and can lose their ability to function in this capacity over time. Therefore, evaluation of soil fertility is important in maintaining an effective bioretention system. Substances in runoff such as nutrients and metals eventually disrupt normal soil

functions by lowering the cation exchange capacity (CEC) (Dept. of Env. Resources, 1993). CEC is the soil's ability to adsorb pollutant particles through ion attraction and will decrease over time. It is recommended that soils be tested annually and replaced when soil fertility is lost. Depending on environmental factors, this usually occurs within 5-10 years of construction. Replacement of soil can be accomplished in 1-2 days for approximately $1,000-$2,000 for a typical system which will drain one acre in the northeastern U.S. (Weinstein, 2000).

2.2 Grass Swales

Grass swales or channels are adaptable to a variety of site conditions, are flexible in design and layout, and are relatively inexpensive (USDOT, 1996). Generally open channel systems are most appropriate for smaller drainage areas with mildly sloping topography (Center for Watershed Protection, 1998). Their application is primarily along residential streets and highways. They function as a mechanism to reduce runoff velocity and as filtration/infiltration devices. Sedimentation is the primary pollutant removal mechanism, with additional secondary mechanisms of infiltration and adsorption. In general grass channels are most effective when the flow depth is minimized and detention time is maximized. The stability of the channel or overland flow is dependant on the erodibility of the soils in which the channel is constructed (USDOT, 1996). Decreasing the slope or providing dense cover will aid in both stability and pollutant removal effectiveness.

Engineered swales are less costly than installing curb and gutter/storm drain inlet and storm drain pipe systems. The cost for traditional structural conveyance systems ranges from $40–$50 per running foot. This is two to three times more expensive than an engineered grass swale (Center for Watershed Protection, 1998). Concerns that open channels are potential nuisance problems, present maintenance problems, or impact pavement stability can be alleviated by proper design. Periodic removal of sediments and mowing are the most significant maintenance requirements.

2.3 Vegetated Roof Covers

Vegetative roof covers or green roofs are an effective means of reducing urban stormwater runoff by reducing the percentage of impervious surfaces in urban areas. They are especially effective in older urban areas with chronic combined sewer overflow (CSO) problems, due to the high level of imperviousness. The green roof is a multilayered constructed material consisting of a vegetative layer, media, a geotextile layer and a synthetic drain layer. Vegetated roof covers in urban areas offer a variety of benefits, such as extending the life of roofs, reducing energy costs and conserving valuable land that would otherwise be required for stormwater runoff controls. Green roofs have been used extensively in Europe to accomplish these objectives. Many opportunities are available to apply this LID measure in older U.S. cities with stormwater infrastructures that have reached their capacities.

Green roofs are highly effective in reducing total runoff volume. Simple vegetated roof covers, with approximately 3 inches of substrate can reduce annual runoff by more than 50 percent in temperate climates (Miller, 2000). Research in Germany shows that the 3-inch design offers the highest benefit to cost ratio. Properly designed systems not only reduce runoff flows, but also can be added to existing rooftops without additional reinforcement or structural design requirements. The value of green roofs for reducing runoff is directly linked to the design rainfall event considered. Design should be developed for the storm events that most significantly contribute to CSOs, hydraulic overloads and runoff problems for a given area.

2.4 Permeable Pavements

The use of permeable pavements is an effective means of reducing the percent of imperviousness in a drainage basin. More than thirty different studies have documented that stream, lake and wetland quality is reduced sharply when impervious cover in an upstream watershed is greater than 10%. Porous pavements are best suited for low traffic areas, such as parking lots and sidewalks. The most successful installations of alternative pavements are found in coastal areas with sandy soils and flatter slopes (Center for Watershed Protection, 1998). Permeable pavements allow stormwater to infiltrate into underlying soils promoting pollutant treatment and recharge, as opposed to producing large volumes of rainfall runoff requiring conveyance and treatment. Costs for paving blocks and stones range from $2 to $4, whereas asphalt costs $0.50 to $1 (Center for Watershed Protection, 1998).

2.5 Other LID Strategies

Another strategy to minimize the impacts of development is the implementation of rain gutter disconnects. This practice involves redirecting rooftop runoff conveyed in rain gutters out of storm sewers, and into grass swales, bioretention systems and other functional landscape devices. Redirecting runoff from rooftops into functional landscape areas can significantly reduce runoff flow to surface waters and reduce the number of CSO events in urban areas. As long as the stormwater is transported well away from foundations, concerns of structural damage and basement flooding can be alleviated. As an alternative to redirection of stormwater to functional landscape, rain gutter flows can be directed into rain barrels or cisterns for later use in irrigating lawns and gardens. Disconnections of rain gutters can effectively be implemented on existing properties with little change to present site designs.

Many strategies exist to reduce the amount of impervious surface in development areas. Designing residential streets for the minimum required width needed to support traffic, on-street parking and emergency service vehicles, can reduce imperviousness. Other practices include shared driveways and parking lots, alternative pavements for overflow parking areas, center islands in cul-de-sacs, alternative street designs rather than traditional grid patterns and reduced setbacks and frontages for homes.

3 EVALUATION OF LID EFFECTIVENESS

3.1 Hydrological Measures

Enhancements in site drainage from traditional stormwater control measures, such as curbs and gutters that eliminate potential on-site flooding, often result in an increase in surface runoff. These alterations can cause an increase in volume, frequency and velocity of runoff flows, resulting in flooding, high erosion and a reduction in groundwater infiltration, as well as a reduction in water quality and habitat degradation. Four hydrological functions should be considered when investigating the effectiveness of LID practices. The runoff curve number (CN), time of concentration, retention and detention. LID techniques and the hydrological design and analysis components are represented in (Table 2).

Table 2: Low Impact Hydrologic Design and Analysis Components (Coffman, 2000)

	Low Impact Hydrologic Design and Analysis Components			
LID Practice	Lower Post-Development CN	Increase Tc	Retention	Detention
Flatten Slopes		X		
Increase Flow Path		X		
Increase Roughness		X		
Minimize Disturbances	X			
Flatten Slopes on Swale		X		X
Infiltration Swales	X		X	
Vegetative Filter Strips	X	X	X	
Disconnected Impervious Areas	X	X		
Reduce Curb and Gutter	X	X		
Rain Barrels		X	X	X
Rooftop Storage		X	X	X
Bioretention	X	X	X	
Revegetation	X	X	X	
Vegetation Presentation	X	X	X	

The runoff potential for a site is characterized by the runoff curve number or CN. One method of measuring hydrological function on a developed site is to compare the pre and post developed curve number. The CN method is used extensively in the analysis of environmental impact and design rainfall-runoff hydrology. The curve number measures a watershed or subwatershed's hydrological response and is determined based on soil type, land cover and amount of impervious surfaces (Hawkins 1998). A detailed evaluation of both proposed and existing land cover is the basis for determining the low-impact development CN, which is a calculation of the potential for runoff at a development site. One of the goals of LID is to design a system so that the post-developed CN is as close as possible to the predevelopment CN for the site. Limiting the percent of imperviousness is one technique to accomplishing this. The runoff coefficient, which can be derived from the CN, calculates the percent of rainfall converted to runoff.

The time of concentration (Tc) refers to the amount of time it takes for water to travel from the most distant point to the watershed outlet. By retaining predevelopment Tc, negative impacts associated with development can be reduced. Retention and detention of rainfall are the key components of increases in Tc. As the amount of impervious surface increases within a site, altering drainage paths, the contribution of total land area to excess rainfall increases, causing the time for stormwater to reach downstream outlets to decrease. This decrease in Tc reduces the pollutant removal capabilities of the site as well as resulting in an increase in the peak runoff rate. Maintaining Tc can be achieved by:

- Maintaining flow path lengths
- Increasing surface roughness
- Detaining flows
- Minimizing disturbances at the site
- Flattening grades in impact areas
- Disconnecting impervious surfaces
- Connecting pervious surfaces

3.2 Pollutant Removal Measures

Changes in site runoff characteristics can contribute to a reduction in water quality and degradation of aquatic and terrestrial habitats. LID practices provide a high level of water quality treatment controls due to runoff volume control of the "first flush" (first ½ inch) of runoff, which contains the highest pollutant loadings. Often LID practices control up to the first 2 inches of runoff and therefore treat a much greater volume of annual runoff (Coffman, 2000). By increasing the Tc and decreasing the flow velocity, LID practices result in a reduction in pollutant transport capacity and overall pollutant loading. Further, LID practices support pollution prevention by modifying human activities, which lower the introduction of pollutants into the environment.

LID practices such as bioretention facilities or rain gardens can be used as a mechanism for infiltration and pollutant removal, which is performed through physical and biological treatment processes occurring in the plant and soil complex. These processes include filtration, decomposition, ion exchange, adsorption and volatilization (Dept. of Env. Resources, 1993). Pollutant loadings are concentrated in the "first flush" of runoff from impervious surfaces and contain grease and oil, nutrients (nitrogen and phosphorous), sediments and heavy metals. Pollutant loadings and water quality impacts from development have been well documented in numerous studies. Concentrations of pollutants are appropriate to look at bio affects, but pollutant loads are better for assessing impacts to downstream habitats when cumulative effects are considered (Rushton, 1999). Studies should consider investigating both total metals and dissolved metals, when analyzing LID practice's effectiveness.

4 CASE STUDIES

The LID "functional landscape" is designed to mimic the predevelopment hydrological conditions through runoff volume control, peak runoff rate control, flow frequency/duration control and water quality control. Determining effectiveness of LID practices can be achieved by evaluating hydrological function and pollutant removal capabilities. Little investigation has been done to prove the actual effectiveness of LID in retaining predevelopment hydrology and preventing or reducing pollutant loadings caused by stormwater runoff on developed sites. LID is a relatively new concept in stormwater management and not widely implemented in all areas and climates in the United States. Limited research and analysis has been conducted on the various practices, due to this limited application.

The following case studies, though limited, represent the best examples of projects that use LID concepts for stormwater management. Both hydrologic and pollutant removal effectiveness are investigated. The most significant source for data is Prince George's County, Maryland where many of the LID practices were developed and first implemented. The Low-Impact Development Center, also located in Maryland, has done significant work in design and planning of LID sites. First year data from a two-year study of a Tampa, Florida, retrofit parking lot and an on-going permeable pavement project in Washington state provide the only long term analysis for the effectiveness of LID concepts (permeable pavements and swales) currently available.

4.1 Bioretention Facility
Laboratory and Field Study
Beltway Plaza Mall Parking Lot, Greenbelt, MD

Introduction

Land development results in increased stormwater runoff at the expense of infiltration. Additionally, surface runoff contains a broad range of pollutants and has been identified as one of the major sources for pollution of natural waters. Detention basins are commonly used for stormwater quality improvement and to optimize the infiltration of stormwater for recharge. A simple, yet effective method to control stormwater is through the use of bioretention areas or rain gardens.

Bioretention systems generally require less space, are more economical to build and require less maintenance than large-scale detention ponds. In addition these landscaped areas have aesthetic value. The design capacity for the system is generally for a typical storm event (0.5-0.7 inches per hour of rainfall over six hours) and to handle runoff from a small development area. The goal of this study is to compare field results with baseline data obtained through a laboratory constructed and tested bioretention systems.

Study Site

This study was conducted in two phases. The first phase took place at the University of Maryland, Department of Civil Engineering, Stormwater Lab in College Park, Maryland. Two different-sized bioretention prototypes were constructed and fitted with ports at varying depths in order to collect and analyze water quality and infiltration data. The small prototype was 2.5 ft wide and 3.5 ft long with a depth of 24 inches of material. The small bioretention system was fitted with two port depths. The large prototype was 10 ft long, 5 ft wide with a depth of 36 inches, and was fitted with three ports at various depth levels. Both systems had a freeboard of 6 inches, to allow water to accumulate if necessary. The soil, organic mulch layer and vegetation, were analyzed prior to construction to assure that the system was constructed according to design recommendations. Simulated runoff was applied to both systems at a rate of 1.6 inches per hour for six hours. A total of 16 simulations were tested on the small box, and four on the large prototype. The total volume of runoff applied to the small system was 200 L, and 1,000 L for the large system. These volumes represent the bioretention prototypes occupying 5% of a drainage area.

The second phase, a field study, took place at an existing bioretention facility located in the parking lot of Beltway Plaza in Greenbelt, Maryland. The depth of the system is 42" and is designed so that runoff infiltrates through the system and is collected by a 6-inch diameter perforated pipe underdrain, which feeds into the main storm drain system. A 7.5-ft x 7.5-ft area of the bioretention facility was used to conduct the study. Approximately 1,000 L of synthetic runoff, with characteristics similar to those used in the laboratory, were applied to the system over a 6-hour period. Effluent samples were collected from the main storm drain at 25-30 minutes intervals.

Study Results Summary

The laboratory results for the smaller prototype showed overall that the removal of heavy metals by the system was good. Cooper, lead and zinc levels in both upper and lower effluents had removal of more than 90%. Copper removal from samples taken from both ports was 94%. Lead removal was more effective from lower ports at 98%, but still good from upper ports at 94%. The average zinc removal from upper and lower ports was >96% (Table 3). No major variation of removal of metals occurred over time and all samples were less than EPA standards for freshwater. Nutrient removal for phosphorous was 65-75% from lower ports and approximately 40% from upper ports. The Total Kjeldahl Nitrogen (TKN) removal is 45-60% for the upper ports and 65-80% for the lower ports. Ammonium and nitrate removal followed no pattern and ranged from zero to 90%.

Table 3: Summary of Results for Smaller System—Standard Conditions

	Cu	Pb	Zn	P	TKN	NH_4^+	NO_3^-	Tn
Removal Upper	94%	94%	97%	25%	55%	60%	11%	60%
Removal Lower	94%	98%	98%	83%	80%	83%	26%	75%

Results from the large prototype correlated with those of the smaller constructed system. Experimental results indicated that removal of metals in most cases was more than 90%. Average copper removal for upper ports was 90% and 93% for middle and lower ports. Lead removal from upper ports was 93%, and >97% for middle and lower ports. The removal of zinc was 87% for upper ports and >96% for middle and lower ports. The data showed a trend of greater metal removal with depth. Nutrient removal was better from lower ports in most cases compared to removal of middle and upper ports. Phosphorous removal for lower ports was about 70-80% and 50-60% for middle ports. The upper ports showed a 10-15% increase in phosphorous levels above the influent amounts. The TKN removal was 50-75% for the lower and middle ports and a 45-30% increase was noted for upper ports. Removal of ammonium was 54% at upper ports, 86% for middle ports and 79% at lower ports (Table 4). Doubling or halving the influent pollutant levels during the laboratory testing had little effect on the effluent pollutant levels. Higher levels of phosphorous and TKN in effluent at the upper ports can be attributed to the vegetation.

Table 4: Summary of Results for Large System—Standard Conditions

	Cu	Pb	Zn	P	TKN	NH_4^+	NO_3^-	TN
Removal Upper	90%	93%	87%	0%	37%	54%	(-97%)	(-29%)
Removal Middle	93%	>97%	>96%	73%	60%	86%	(-194%)	0%
Removal Lower	93%	>97%	>96%	81%	68%	79%	23%	43%

During the field test at Beltway Plaza, a total of 1,000 L of synthetic runoff were applied to the bioretention area over a 6 hour period at a rate of approximately 0.5 inches per hour. Of the 1,000 L of influent, only 39% left the system. The remaining water leaked through cracks into the manhole, was held in the facility, or infiltrated. Effluent samples were analyzed for removal of nutrients and heavy metals (copper, lead and zinc).

The TKN removal was about 50% and the phosphorous removal was observed at approximately 65%. Nitrate concentrations were below input levels, with a removal of about 17%. The removal for ammonia was very good at >95%. Removal of metals was very good and was consistent with the laboratory results. The removal of copper was 97% and for lead, and zinc, the removal was >95% (Table 5).

Table 5: Summary of Results for Field Bioretention Study

	Cu	Pb	Zn	P	TKN	NH_4^+	NO_3^-	TN
Removal	97%	>95%	>95%	65%	52%	92%	16%	49%

Removal rates for the field study corresponded with the rates observed for the two laboratory constructed bioretention systems. In all cases pollutant removal rates approached 100% for the metals copper, zinc and lead. Doubling or halving the concentration levels of the influent had no effect on removal efficiencies and were statistically equivalent in nearly all cases. Pollutant removal rates for all systems are compared in the above graph (Figure 3). The negative removal rate for nitrate in the large prototype, upper and middle ports, was attributed to the release of previously captured nitrated or nitrate from nitrification processes.

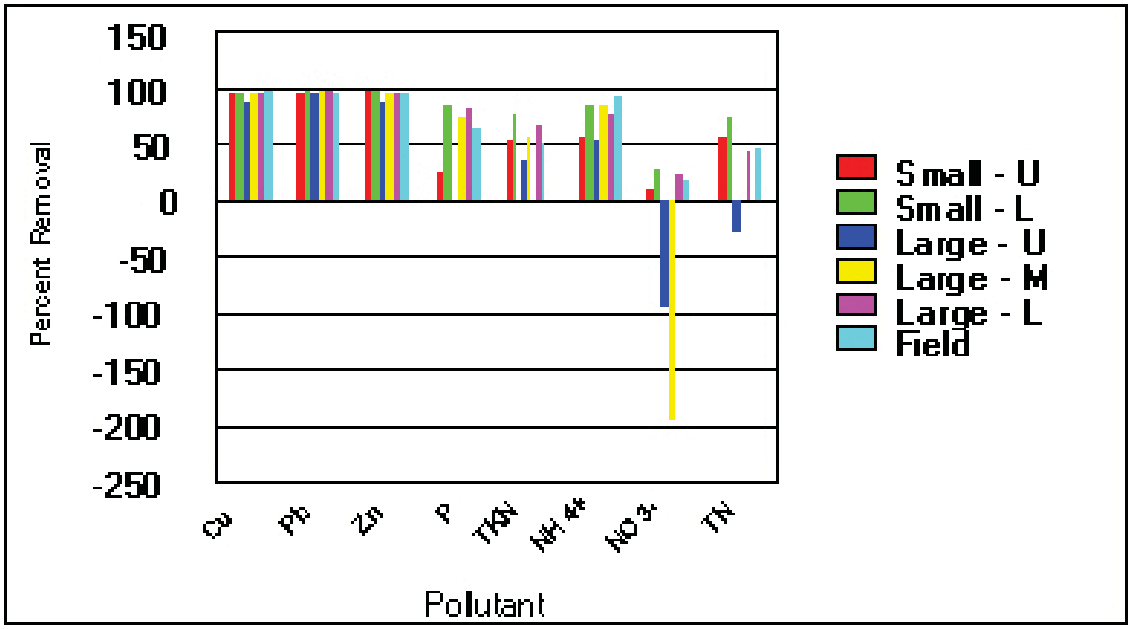

Figure 3: Pollutant Removal Rates for All Systems

4.2 Bioretention Facility Field Study Peppercorn Plaza Parking Lot at Inglewood Center, Landover, MD

Introduction

Impervious surfaces, such as parking lots, are a major contributor to pollutant loads in receiving waters in urban areas. These surfaces provide a place for pollutants to accumulate and later wash-off in the first flush of rainfall events. Parking lots are good site locations for bioretention systems, since they can be retrofit into existing lots with little or no loss of parking space. In addition, patrons have expressed appreciation of green space within parking areas. Bioretention areas are a natural means of controlling pollutants from entering urban water bodies. The hydrologically functional landscape, can be used as a mechanism for pollutant removal, through physical and biological treatment processes occurring in the plant and soil complex. The bioretention area in the Inglewood Center Parking lot, was analyzed for pollutant removal efficiency during a simulated rainfall event.

Study Site

The study was conducted at one of the two bioretention areas in the Inglewood Plaza parking lot. An area of 50 ft^2 was used in the south facility for the simulated rainfall event. The bioretention facility contains a T-shaped under drain that runs the entire length of the system and is located 32.5 inches below the surface (Figure 4). The under drain directly connects with the storm drainage system. Samples were collected from a pool of water in the storm drain observation area. Output samples were collected every 30 minutes. The soil was dry at the onset of the experiment, due to lack of rainfall for a period of several days prior to the experiment. The synthetic rainfall was applied at a rate of 1.6 inches per hour for a duration of six hours. A total of 300 gallons (1100L) was applied over the course of the experiment.

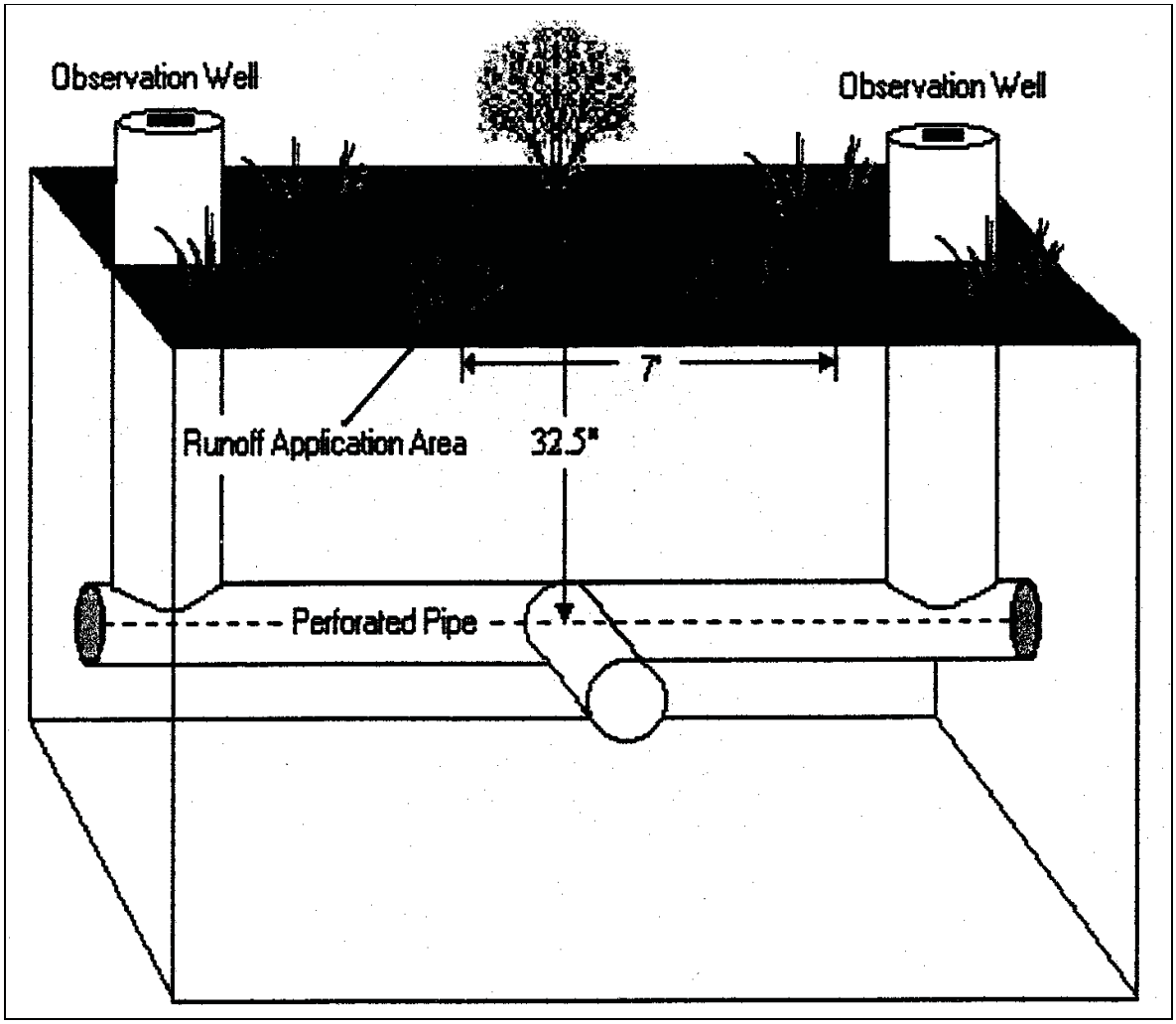

Figure 4: Bioretention System at Peppercorn Place, Inglewood Plaza (Davis, 1999)

Project Results Summary

Effluent concentrations for metals were fairly constant over the sampling period, with zinc being the exception by showing improved removal over time. Average removals for total copper was 43%, total lead was 70% and total zinc 64%. The removals were 5–14% better for dissolved metals. Nutrient concentrations were all below input levels. Removal of phosphorous was very good at 87%. Removal of TKN was observed at 67% and nitrate averaged 15% (Table 6). Ammonium was not detected in either the influent or the effluent. In addition, the bioretention facility removed some calcium, however chloride concentrations were higher in the effluent than in the influent, which is attributed to salting of the parking lot in the winter. Also, temperature variations during the experiment showed evidence of the system cooling the runoff water temperature.

Table 6: Summary of Pollutant Removal Results of Bioretention System at Inglewood Place

	Cu	Pb	Zn	Ca	P	TKN	NO_3^-
Removal	43%	70%	64%	27%	87%	67%	15%

By using synthetic runoff, the concentrations of applied pollutants could be controlled and accurately measured and compared to levels found in the effluent. However, testing has not been done on an actual rainfall event to determine effectiveness of the system for reducing runoff volume and pollutant loads.

4.3 Permeable Pavements and Swales Field Study Stormwater Management, Florida Aquarium Parking Lot, Tampa, FL

Introduction

Impervious surfaces are responsible for more stormwater runoff than any other type of land use. Paved surfaces that often replace vegetated areas increase the volume and frequency of rainfall runoff. In addition, these surfaces provide a place for pollutants to accumulate between rainfall events, and are later washed off into receiving waters. Keeping runoff on-site to allow for infiltration as well as chemical, physical and biological processes to take place is the most effective means of reducing pollutant loadings. This study quantifies how much runoff and pollutant loadings can be reduced by using swales and landscaped depressions in parking lots. In additional to investigating basins with and without swales, three paving surfaces were compared. The research is designed to determine pollutant load reductions measured from three different treatments within the parking lot; different paving materials in the parking lot, a planted strand with native trees and a small pond used for final treatment. Pollutant concentrations and infiltration were measured and analyzed for the various control methods. First year data collected in the parking lot between August 1998 and August 1999 were evaluated for this study. Also, sediment samples were collected from each of the swales, two locations in the strand and two locations in the pond.

Project Area

The study site is a parking lot at the Florida Aquarium in Tampa, Florida. The study uses the entire parking area, 4.65 ha, to define the drainage basin. The parking lot was modified for the study by reducing the length of each parking space by 61 centimeters, which allows for a 122-cm wide grass swale between rows. The vehicle front end now hangs over a grass swale instead of pavement, which prevented any reduction in the number of parking spaces within the parking area. Four different scenarios were investigated to determine the most efficient method of runoff reduction and pollutant removal. Eight basins, two of each type, were constructed and fitted with instrumentation to collect flow weighted water quality samples and measure discharge amounts during storm events (Figure 5). The four treatment types are:

- Asphalt paving with no swale
- Asphalt paving with a swale
- Cement paving with a swale
- Permeable pavement with a swale

Rainfall quality and volume were compared to runoff quality and volume to determine the effectiveness of each treatment type.

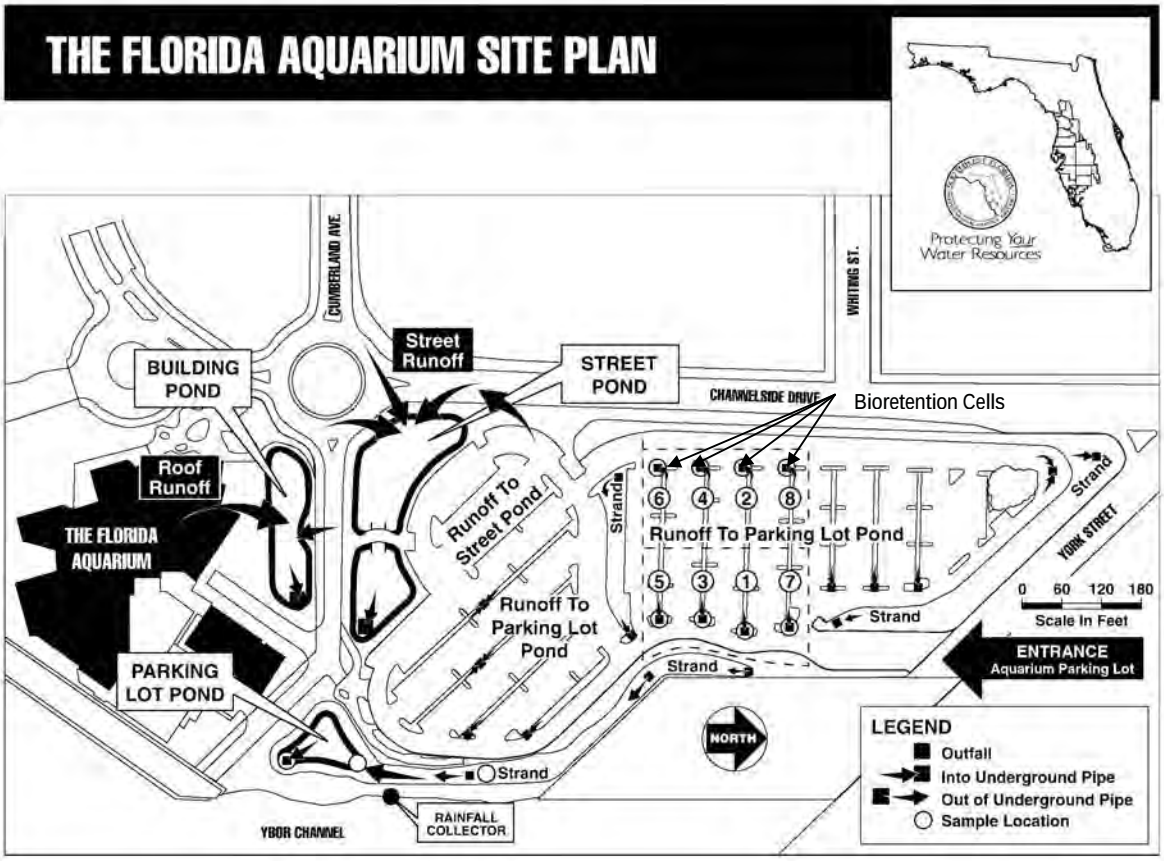

Figure 5: Florida Parking Lot Study Site (Rushton, 1999)

Project Results Summary

The larger garden areas (approximately the size of one parking space) account for a runoff coefficient calculation reduction of 40-50 percent for the smaller basins. The runoff coefficient is a value that ranges from zero to one and expresses the fraction of rainfall volume that is actually converted into storm runoff volume. The runoff coefficient closely tracks percent impervious cover. For rainfall events less than 2 cm, basins with swales and permeable pavement have 80-90% less runoff than basins without swales, and 60-80% less runoff than basins with the other pavement types and swales. The percent of rainfall converted to runoff for each treatment type is shown in Figure 6.

Larger rainfall amounts show fewer differences in runoff amounts between the different pavement types, but basins with swales have approximately 40% less runoff than the basins without swales. Soil analysis at the site shows a higher than average gravel content (8.9%) which may account for the good infiltration rates. Comparisons of rainfall with storm runoff amounts showed that swales reduced runoff for all rainfall events and paving types.

Water quality analysis shows that average concentrations varied by paving and depression storage types. Rainfall has been identified in other studies as a significant source of nitrogen in runoff. This site displayed the same correlation between

concentrations of ammonia and nitrate in rainfall and their concentrations in runoff. Phosphorous concentrations displayed the inverse, since concentrations were higher in effluent samples than in the initial rainfall. The levels were somewhat higher in the runoff of basins without planted swales and the highest concentrations of phosphorous were noted in basins where runoff traveled through grassed swales.

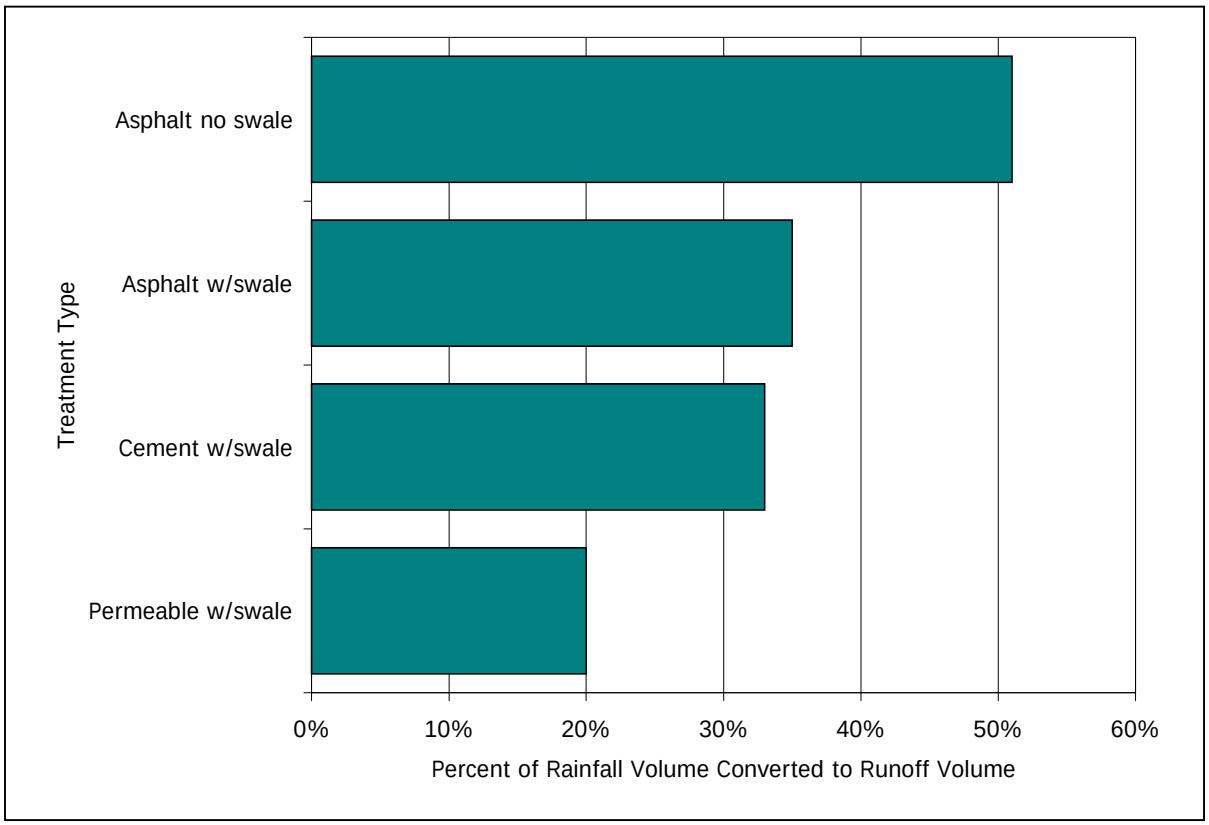

Figure 6: Percent of Rainfall Volume Converted to Runoff Volume for Events Less Than 2cm

Paving material showed an effect on the concentration of metals in runoff. Basins paved with asphalt showed higher concentrations of iron, manganese, lead, copper and zinc than those paved with cement or permeable paving. Many of the major ions also showed a correlation with the paving material. Potassium, sodium, sulfate and calcium concentration were much higher in the basins paved with cement, which is made from limestone, although these levels were still well below levels considered detrimental to the environment. No consistent pattern was discernable for suspended solids, but generally measurements were low when compared to similar stormwater studies.

Water quality loads were examined because they provide a more realistic measure for understanding the impacts of stormwater on receiving waters. Pollutant loads include both the volume of water discharged and the concentration of pollutants measured. Higher loads for all constituents, except phosphorous, were noted for basins without swales, since more water was discharged from these basins. Although phosphorous concentrations were much lower in basins without swales, loads were about the same.

Removal for Ammonia was 45% for asphalt with swale, 73% for cement with swale and 85% for permeable pavement with swale. Total nitrogen removal was 42% for permeable pavement with swale, 16% for cement with swale and 9% for asphalt with swale. TSS removal varied from 91% for permeable pavement with swales to 46% for asphalt with swales.

Table 7 summarizes the constituent load efficiency of the various treatments. The concentrations and loads measured during this study were compared to other stormwater studies conducted in Florida, and the values were much lower than measured values at other sites. Metal removal was good for the permeable pavement with swale treatment, with copper at 81%, iron 92%, lead 85%, manganese 92% and zinc 75%. The removals for the cement with swale treatment were somewhat lower, with the asphalt with swale treatment showing the poorest performance of the three treatments with swales.

Table 7: Summary of Pollutant Removal Efficiency for the Various Treatment Types

Constituent	Asphalt with swale	Cement with swale	Permeable with swale
Ammonia	45%	73%	85%
Nitrate	44%	41%	66%
Total Nitrogen	9%	16%	42%
Ortho Phosphorus	-180%	-180%	-74%
Total Phosphorus	-94%	-62%	3%
Suspended Solids	46%	78%	91%
Copper	23%	72%	81%
Iron	52%	84%	92%
Lead	59%	78%	85%
Manganese	40%	68%	92%
Zinc	46%	62%	75%

The concentrations of metals in sediment samples collected in swales were consistent with concentrations measured in stormwater runoff. Higher concentrations of metals were found in swales paved with asphalt than those of grass. None of the metals measured in the sediments exceed the level where toxicity to organisms is probable when compared to the Environmental Protection Agency (EPA) and the National Oceanic and Atmospheric Administration (NOAA) chemical toxicity guidelines for marine environments. However, copper and zinc concentrations were above the level where toxicity is possible.

Nutrient concentrations measured in sediment samples for TKN and total phosphorus were lower in the basins without grassed swales. Sediment samples taken from locations in the strand and the wet-detention pond were compared to swale samples. The comparison showed that most of the metals are being settled out in the swales or

deposited in the drop boxes. Sediment samples at the site were tested for 100 organic pollutants, but only 16 were detected at the site. The high concentrations found in this and similar studies indicate that atmospheric deposition is the source for most of the 16 detected organic pollutants.

4.4 Vegetated Roof Covers
Field Study
Green Rooftop, Philadelphia, PA

Introduction

Many older American cities are plagued with nuisance flooding on roads and walkways and chronic overflows of combined sewer systems. In highly impervious cities, vegetated rooftops offer a practical solution for controlling runoff at the source. A vegetated roof cover is a veneer of living vegetation installed on top of a conventional roof. By mimicking natural hydrologic processes, they can achieve runoff characteristics similar to open space conditions. Green roofs are comprised of three components; subsurface drainage, growth media and vegetation. Specific hydraulic performance objectives are achieved through the appropriate selection of these components. Vegetated roof covers have been used extensively in Germany for 25 years.

Project Area

A 3,000-ft^2 rooftop in Philadelphia was fitted with a demonstration vegetated rooftop. The performance objective was the restoration of predevelopment runoff peak rates for a 24-hour, 2-year return-frequency storm. Although in the Philadelphia area, 90% of all rainfall is contributed by storms with volumes of 2 inches or less over a 24-hour period. The "green roof" used is only 3.4 inches (8.6cm) thick, including the drain layer (Figure 7). Its maximum saturated weight is less than 17 lb/ft^2 and it weighs less than 5lb/ft^2 when dry. No additional structural support was necessary for installation. The saturated infiltration capacity is 3.5 inches per hour. The key features of this system are a synthetic under drain layer which promotes rapid water drainage from the roof surface, thin, lightweight growth media suitable for installation on existing roof surfaces and a meadow-like setting of perennial *Sedum* varieties selected for hardiness and the ability to withstand seasonal conditions typical of the area.

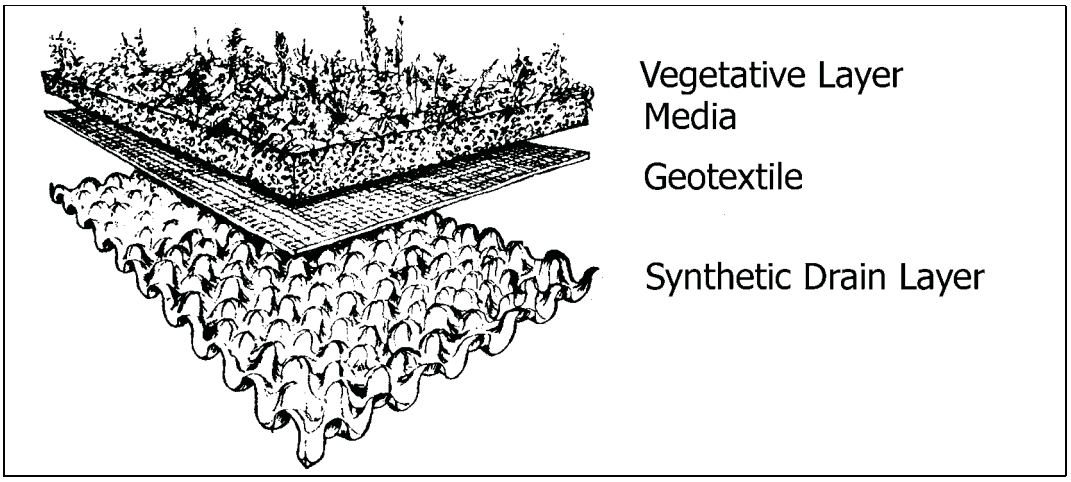

Figure 7: Structure of the Philadelphia Vegetated Roof Cover (Miller, 1998)

Project Results Summary

Currently too few storms have been observed to permit quantitative assessment of the vegetative covered roof. Data are available from one intense storm monitored during a 0.4 inch, 20-minute rainfall event (Figure 8). Supplemental data from a pilot-scale experimental station were used in this study. Test data show that for storms with less than 0.6 inches, runoff is negligible. During a 9-month period, 44 inches of rainfall was recorded at the pilot-scale test station, with only 15.5 inches of runoff generated. Runoff occurred for precipitation events between 0.6 and 1.0 inches, but lagged rainfall significantly. Attenuation was lower for the pilot-scale experiment than the anticipated modeled value (40% vs. 48%), which has been attributed to differing drain conditions and a steeper slope at the test site. Additional benefits of this project include extended life of the underlying roof materials, reduction of energy costs by improving effectiveness of insulation and restoration of ecological aesthetic value of open space in densely populated areas.

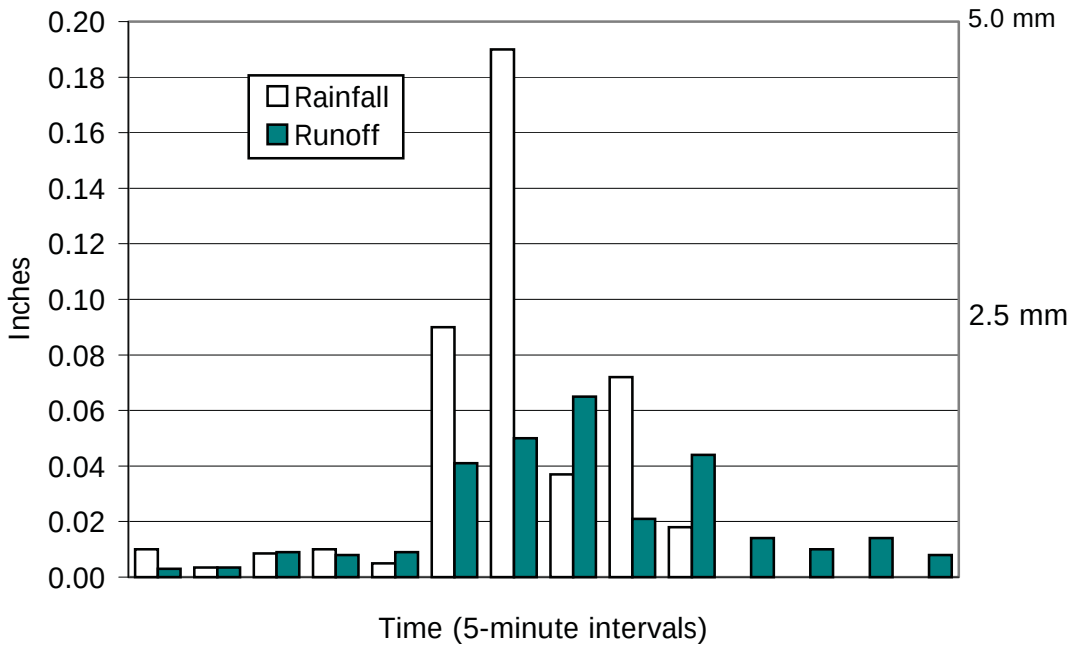

Figure 8: A Rainfall Event of 0.4 inches with Media Completely Saturated (Miller, 1998)

4.5 Permeable Pavements
Field Study
Permeable Pavements for Stormwater Management, Olympia, WA

Introduction

This study demonstrates the use of permeable surfaces for reducing runoff volume, improving infiltration and reducing pollutant loadings in an urban parking area. Numerous problems associated with urbanization, such as flooding, channel erosion and destruction of aquatic habitats are directly linked to the loss of water-retaining function of soil in urban landscape. As imperviousness increases, a stormwater runoff reservoir of tremendous volume is removed. Water that may have lingered in this reservoir for anywhere from a few hours to many weeks now flows rapidly across land surfaces and arrives at stream channels in short, concentrated bursts. The scope of this project was to review existing information on types and characteristics of permeable pavements, construct and monitor a full-scale test site and evaluate long-term performance of these systems. This study of permeable pavements evolved from a growing recognition of the limitations of traditional stormwater management in keeping water in the soil by allowing excess of water to the soil over large areas of landscape.

Study Site

The study site is an employee parking lot on the southeast corner of the King County Public Works facility in Renton, Washington. The permeable pavement sections of the lot were constructed for the purpose of this study. A total of eight stalls using four different pavement types were constructed. In addition a ninth stall of traditional asphalt was used as a control. The parking stalls are fitted with pipe, gutters and gauges to collect and measure the quantity and quality of storm runoff from each pavement type. Subsurface troughs were constructed down the middle of each stall and imbedded into the subgrade six to 8 inches below the surface. This allows for the collection of only a fraction of the infiltrated water (about 1.8%). The permeable pavement types studied were:

- A plastic network with grass infilling (<5% impervious)
- An equivalent plastic network with gravel infilling (<5% impervious)
- Impervious blocks with grass infilling (~60% impervious)
- Impervious blocks with gravel infilling (~90% impervious)

Project Results Summary

Data used to monitor the various permeable pavements were from three different storm events during the autumn of 1996. The volume of runoff generated from cement blocks with 60% impervious surface stalls and runoff from traditional asphalt are compared (Figure 9). The storm had a fairly uniform distribution of rainfall (4mm per hour) throughout the duration of the event. Rain falling on the asphalt yielded sharp hydrograph

peaks and a high total volume of runoff water. Only about one peak per hour (0.03mm per hour of runoff) was recorded for the cement blocks with 60% impervious surface. These data are representative of data gathered at the other stalls and reflect little or no runoff from the permeable pavement stalls.

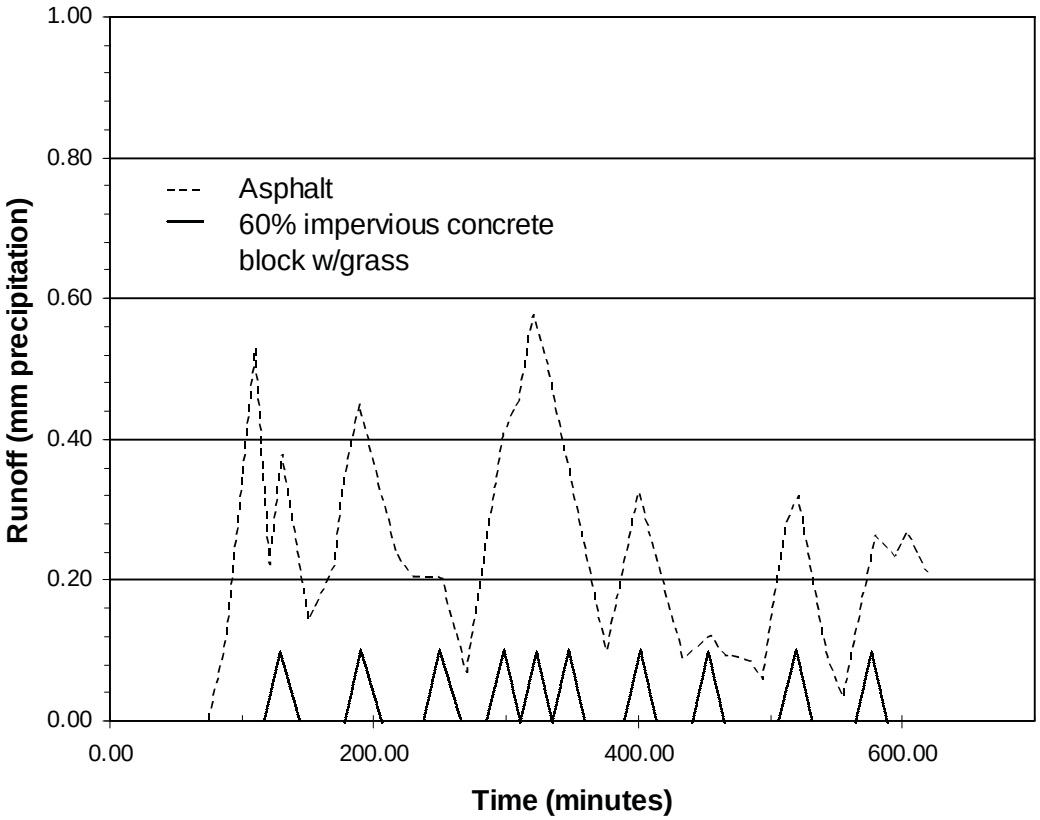

Figure 9: Surface Runoff from 60% Impervious Pavement vs. Asphalt (Booth, 1996)

In contrast to surface runoff, subsurface flow generally responds more slowly and more uniformly. The data for a storm of short duration and moderate intensity are represented in the following graph (Figure 10). Individual peaks on the bar graph indicate rainfall rates as high as 14mm per hour, lasting for short durations (15-minute intervals). Runoff gauges on all four systems showed virtually no surface runoff (on average 0.03 mm). It displays a characteristically attenuated discharge peak and lagged response to the rainfall inputs. All pervious surfaces responded similarly. For the asphalt surface, the volume of water running off the asphalt responded quickly to changes in the rate of rainfall. This is indicated by high peak flows corresponding with precipitation amounts, with little lag time noted (Figure 11).

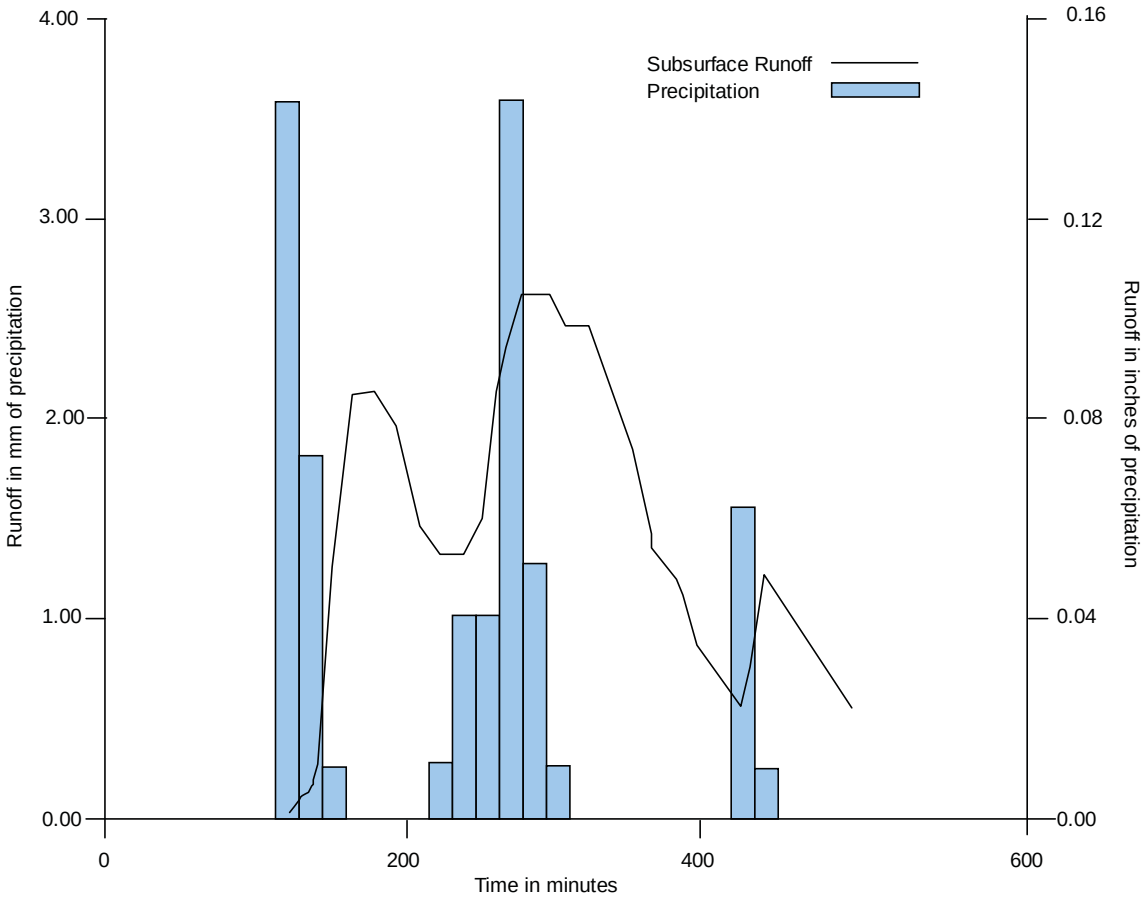

Figure 10: Subsurface Runoff From Pavement Less Than 5% Impervious Compared to Precipitation (Booth, 1996)

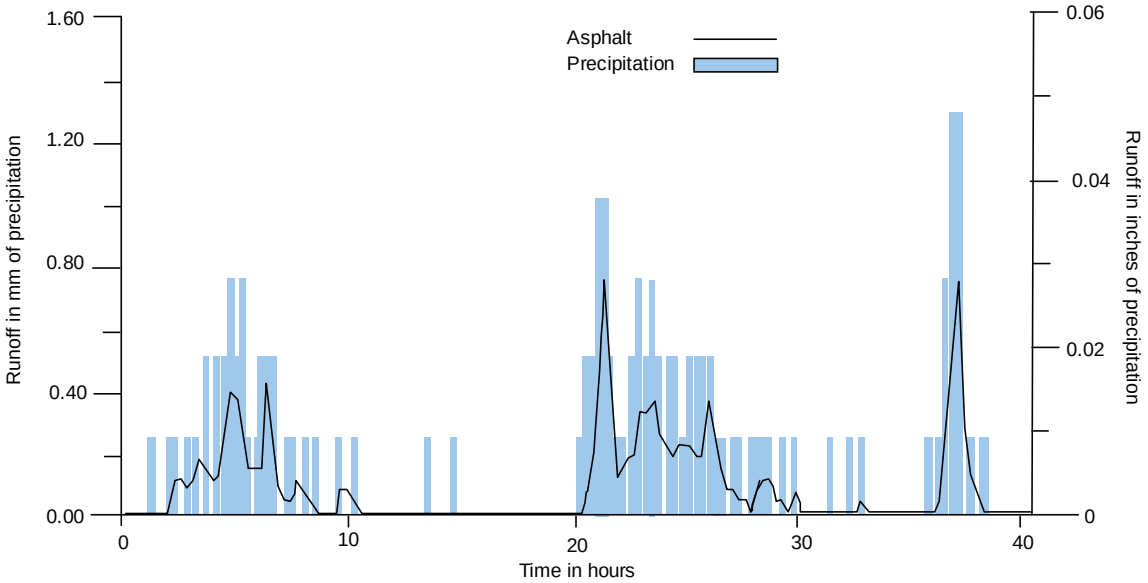

Figure 11: Surface Runoff From Asphalt Compared to Precipitation (Booth, 1996)

Water quality results were obtained from samples collected directly from tipping bucket gauges. Only five samples from the four subsurface collection troughs and the asphalt surface runoff were analyzed. Chemical analysis of the subsurface samples showed sub-detection levels for many of the constituents and relatively low levels for all tested compounds. Measured concentrations of common metals (copper, lead, zinc aluminum and iron) were substantially below the reported national averages. Subsurface samples did show slightly higher concentrations than runoff, which can be attributed to the troughs collecting the "dirtiest" 2 percent of runoff, from directly under where vehicles park. Still, these concentrations were below typical values seen in urban runoff.

4.6 Grass Swales
Field Study
Highway Grass Channels, Northern Virginia, Maryland, and Florida

Introduction

The U.S. Department of Transportation, Federal Highway Administration conducted a field study to determine the pollutant removal efficiencies of grassed channels and swales along highways in Northern Virginia, Maryland and Florida. Sampling was conducted at the inflow and outflow areas of the channels, which provided data for quantity and quality of waters entering and leaving the channels. The samples were analyzed for the following pollutants:

- Total Suspended Solids (TSS)
- Heavy Metals (cadmium, copper, lead and zinc)
- Nitrogen (Total Kjeldahl Nitrogen and nitrite/nitrate)
- Total Phosphorus
- Total Organic Carbon

Twelve rainfall events were monitored, including both frequent and infrequent rainfall periods, most involving discrete stormwater runoff events following a minimum of two days of dry weather. In addition continuous rainfall periods of seven to 14 days were included to determine overall removal efficiencies.

Project Area

The test area in northern Virginia is located along I-66. The channel has an average slope of 4.7% with a total drainage area of 1.27 acres (0.51 ha). Stormwater enters the channel indirectly, by means of overland flow. Stormwater data were collected from June 13, 1987 through November 12, 1987. The test site in Maryland is a grass channel located alongside I-270. This channel has a slope of 3.2% and a total drainage area of 1 acre (0.40 ha) with stormwater entering by means of overland flow. Data were collected for the period beginning June 18, 1987 and ending mid-September 1987. The Florida test site is a grass channel median located between the East and West lanes of I-4. The Florida grass channel has a lower slope than the other two test sites with a drainage area of 0.56 acres (0.23 ha). Data collection began at this site on February 25, 1988 and ended on October 31, 1988.

Project Results Summary

All three locations showed some effectiveness with regard to pollutant removals, although results varied depending on the method of analysis and the location. The results for all three locations are represented in Table 8. Sediment core samples were obtained from the channels and compared to samples from adjacent, upland areas, to determine

pollutant removal effectiveness of the grass channels. Based on the data from the analysis the following conclusions were made. Removal of metals appears to be directly related to the removal of TSS, whereas nutrient removal is not. Removal of TSS can be estimated using flow depth and travel time relationships. Relatively low nutrient removal may be observed in channels that are effective in removing other pollutants. The controlling factors in pollutants removal of grass channels are length, channel geometry, channel slope and average flow. Both metals and nutrients are removed in grass channels, but metal removal is more reliable.

Table 8: Long Term Pollutant Removal Estimates for Grassed Swales

	TSS	TOC	TKN	NO_2/NO_3	TP	Cd	Cr	Cu	Pb	Zn
VA	65%	76%	17%	11%	41%	12-98%	12-16%	28%	41-55%	49%
MD	-85%	23%	9%	-143%	40%	85-91%	22-72%	14%	18-92%	47%
FL	98%	64%	48%	45%	18%	29-45%	51-61%	62-67%	67-94%	81%

5 CONCLUSION

Pollutant loading reduction data for bioretention systems are promising in that removal percentages for heavy metals and nutrients seem quite high. Generally, the experimental data show a fairly consistent removal rate for all of the tested bioretention systems for heavy metals and most nutrients (Table 9). Field study results support the laboratory baseline data collected by the University of Maryland, College Park. However, the field studies provide data for single, simulated rainfall events using synthetic rainfall. A larger number of sampled events would be required for statistical validity of the results.

Table 9: Pollutant Removal Efficiencies for Laboratory and Field Bioretention Studies

Pollutant	Laboratory (small)	Laboratory (large)	Beltway Plaza	Inglewood Plaza
Pb	93–97%	93–97%	>95%	70%
Cu	91–97%	90–93%	97%	43%
Zn	93–98%	87–96%	>95%	64%
P	16–83%	0–81%	65%	87%
TKN	55–80%	37–68%	52%	67%
NH_4^+	<0 -83%	54 -86%	92%	N/A
NO_3^-	11–26%	<0–23%	16%	15%
TN	60–75%	<0–43%	49%	N/A

The use of synthetic runoff during the bioretention experiments, both in the lab and field, allowed the concentrations of applied pollutants to be controlled and accurately measured, so that influent and effluent levels could be compared. In addition, infiltration could be determined based on the volume of runoff verses volume input. The statistical analysis applied for the mass loadings was sound. However, testing for these studies has not been conducted for any actual rainfall events to determine effectiveness of the system for reducing runoff volume and pollutant loads. A comparison of average pollutant removal efficiencies is shown in Figure 12.

The grass swale data from the Federal Highway study show trends in removal of metals as they relate to TSS removal for three different areas in the United States. However a short study period, using data from only a few storm events, is used to quantify the results. Additional data from numerous storm events would be required to provide statistical validity to the analysis. The data from additional, less extensive studies conducted by the University of Virginia help to validate the highway data, as pollutant loading removal rates and runoff volume reduction rates were fairly consistent between the two studies. Conclusions drawn from both studies indicate that not only length, but also longitudinal slope and the presence of check dams increase the pollutant removal capabilities (Kuo, 1999).

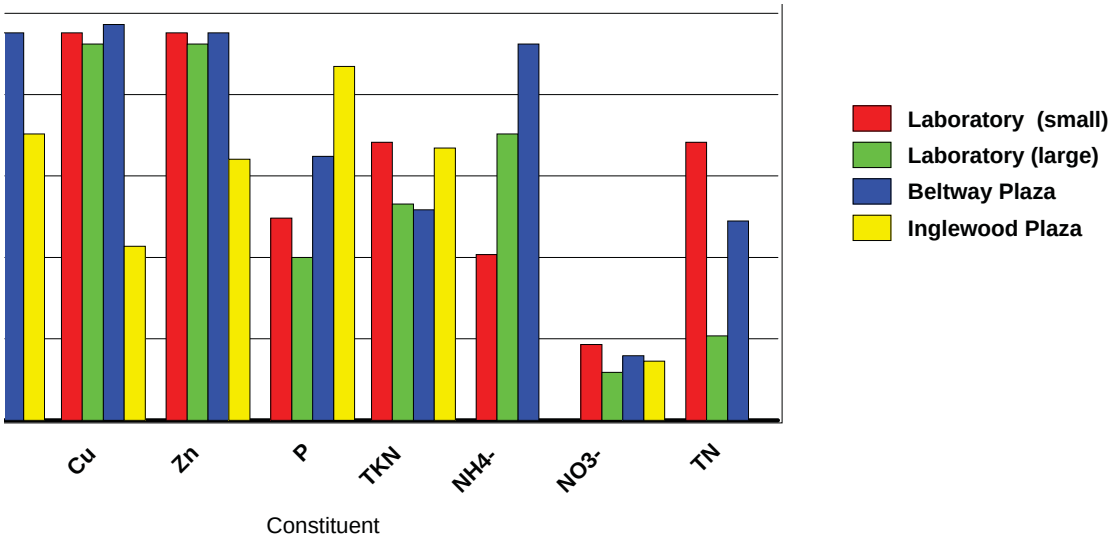

Figure 12: Pollutant Removal Rates for Laboratory and Field Experiments of Bioretention Systems

In addition, a study conducted in Ontario, Canada concluded that no evidence existed to show that nutrient or metal concentrations in soils increased with age in grass swales, as concentrations varied regardless of age. Also, the Canadian study determined that no degradation in vegetative quality resulted from continuous exposure to stormwater runoff. It was shown that vegetation quality was similar to what would be found along conventional systems (Sabourin, 1999). The Canadian study also showed that total runoff volumes from grassed swales were 6-30% less than conventional systems and that a loading comparison revealed that the system released significantly less pollutants than conventional systems.

Permeable pavements can reduce the percent imperviousness for urban areas, which allows for greater infiltration rates and reduced runoff volumes. In addition these alternate pavement types function as stormwater pollutant removal mechanisms. Preliminary data from the Washington project show effectiveness, but too few storms have been analyzed. Only the Florida Aquarium parking lot data represent an analysis of a significant number of actual storm events. As the study continues, and second year data become available, more compelling proof of the pollutant removal effectiveness and runoff volume reduction can be realized. The methodology for testing runoff volume reduction and mass pollutant loadings in the Florida study provided reliable data.

Extensive data exist that show runoff volume reduction using vegetated roof covers in Europe, especially Germany. The data are specific to temperate climates and results may vary considerably for other areas in the United States. However, the Philadelphia project shows the benefits of this application in reducing runoff volume by reducing the level of imperviousness in urbanized areas. Further, it demonstrates the capacity for retrofit of green roofs in highly impervious, older, urbanized U.S. cities experiencing chronic CSO problems. Little data are available from this demonstration project. However, with continued monitoring, evidence of the suitability of green roofs in the United States may become more apparent.

6 RECOMMENDATIONS

A detailed comparison of pre- and post-development conditions and an analysis of adjacent areas using traditional stormwater controls and LID practices side-by-side, would provide the best possible assessment of LID effectiveness hydrologically and as a mechanism for reducing pollutant loadings. The Jordan Cove Urban Watershed project in Waterford, Connecticut, is currently under construction for a side-by-side analysis, however, no data are available at this time. Baseline predevelopment hydrological data are currently being collected for comparison once the development is completed and monitoring begins.

Most of the current field data available for bioretention facilities are for single, simulated rainfall events. Fitting the existing, tested bioretention areas in Prince George's County with monitoring equipment and running a significant number of tests on actual rainfall events over 9 months to 1 year, would provide higher quality data. Long term studies would prove or disprove the long-term effectiveness of bioretention systems, as well as provide information on trends in soil fertility lifetimes and trends in reduced capabilities over time. The two-year Florida Aquarium study is currently the best possible source for these data.

The majority of case studies cited above are ongoing investigations, and reported data represent preliminary findings. Follow-up on these studies will provide better support for proof of effectiveness of LID practices. Additional studies testing LID practices should be identified as the use of these practices grows. Preliminary findings should be viewed as a starting point, and not the empirical proof of effectiveness for the various LID practices studied. The development of a database for entry and storage of LID study data could provide a useful tool for future investigation of LID effectiveness.

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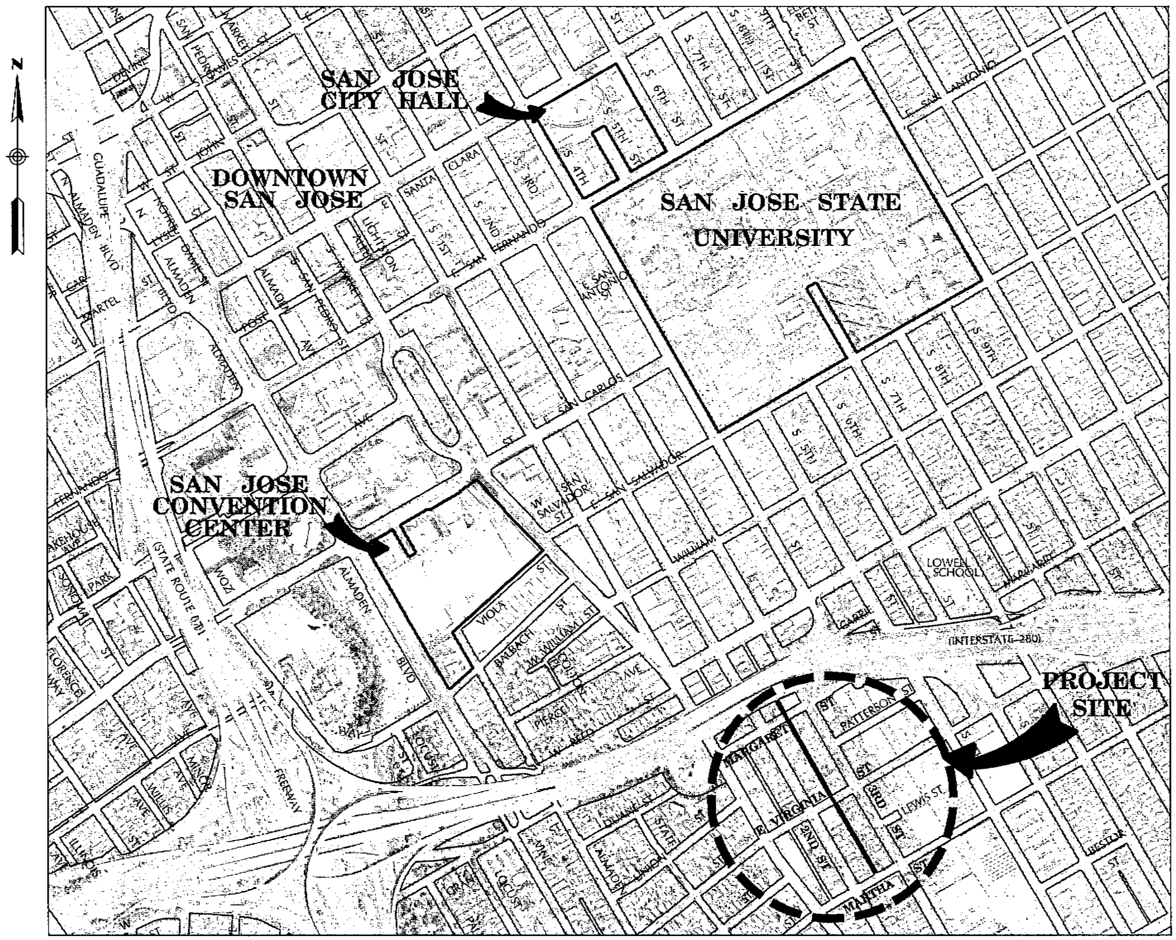
CITY OF SAN JOSE
DEPARTMENT OF PUBLIC WORKS
PLANS FOR:
MARTHA GARDENS GREEN ALLEYS PILOT PROJECT
MARGARET STREET TO MARTHA STREET



LOCATION MAP

NOT TO SCALE

FIGURE 1–3

FIGURE 2–3 SEE SHEET 3

MARTHA GARDENS GREEN ALLEYS PILOT PROJECT
(Between Margaret Street and Martha Street)
CONCEPTUAL PLAN

6		
5		
4		
3		
2		
1		
	REVISIONS	DATE

SANITARY ____ ELECTRICAL ____
STORM ____ CONSTRUCTION ____
MATERIALS LAB ____ TRANSPORTATION ____
SURVEY ____ TRAFFIC OPERATION ____
ROADS ____ MAINTENANCE ____
GEOMETRIC ____

CITY OF SAN JOSE
CAPITAL OF SILICON VALLEY

DEPARTMENT OF PUBLIC WORKS
SAN JOSE, CALIFORNIA

DRAWN BY: C. TORIBIO
CHECKED BY: S. KUMAR / E. YUEN
PROJ MGR: S. KUMAR
DATE: JANUARY 2012
SCALE: 1" = 20'
SHEET NO. 2 OF 4

DAVID SYKES
DIRECTOR OF PUBLIC WORKS

CPMS ID: 6909

MARTHA GARDENS GREEN ALLEYS PILOT PROJECT – CPMS ID 6909

FIGURE 2–3

FIGURE 1–3 SEE SHEET 2

FIGURE 3–3 SEE SHEET 4

MARTHA GARDENS GREEN ALLEYS PILOT PROJECT
(Between Margaret Street and Martha Street)
CONCEPTUAL PLAN

6		
5		
4		
3		
2		
1		
	REVISIONS	DATE

SANITARY
STORM
MATERIALS LAB
SURVEY
ROADS

ELECTRICAL
CONSTRUCTION
TRANSPORTATION
TRAFFIC OPERATION
MAINTENANCE
GEOMETRIC

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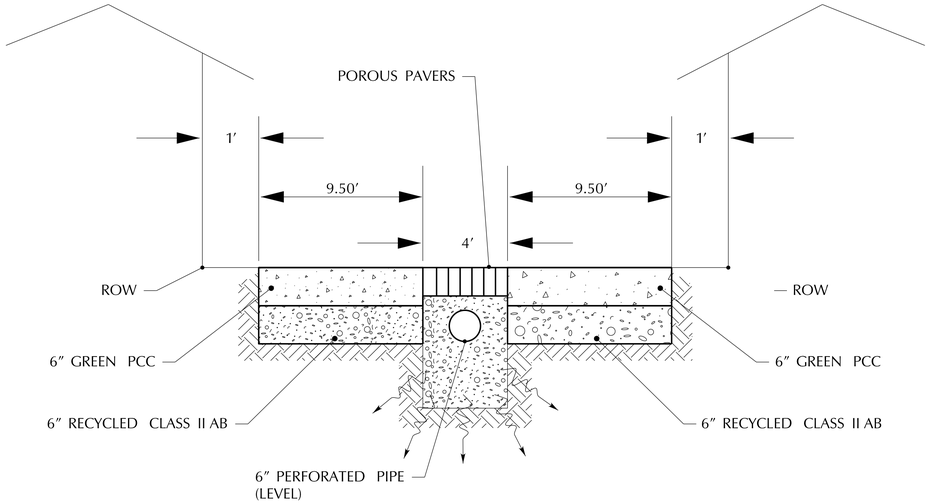


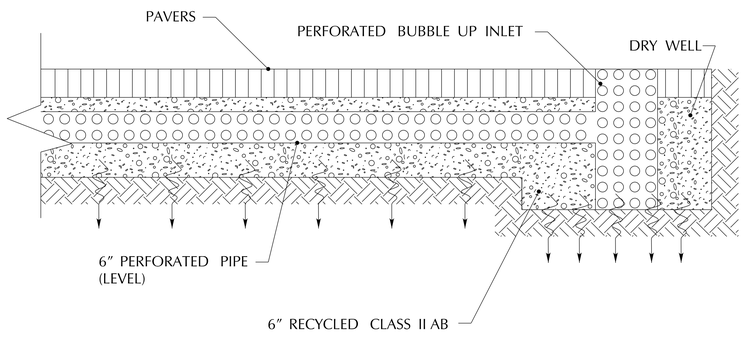
TYPICAL CROSS SECTION
NTS

LEGEND

PERMEABLE PAVERS

NEW GREEN CONCRETE

DRY WELL



PROFILE OF DRY WELL WITH BUBBLE UP SYSTEM
NTS

FIGURE 3–3

FIGURE 2–3 SEE SHEET 3

MARTHA GARDENS GREEN ALLEYS PILOT PROJECT
(Between Margaret Street and Martha Street)
CONCEPTUAL PLAN

	REVISIONS	DATE
6		
5		
4		
3		
2		
1		

SANITARY
STORM
MATERIALS LAB
SURVEY
ROADS

ELECTRICAL
CONSTRUCTION
TRANSPORTATION
TRAFFIC OPERATION
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DAVID SYKES
DIRECTOR OF PUBLIC WORKS

CPMS ID: 6909

MARTHA GARDENS GREEN ALLEYS PILOT PROJECT – CPMS ID 6909

Initial Study/Mitigated Negative Declaration

Shortcut Pipeline Improvement Project

September 22, 2011

CONTRA COSTA
WATER DISTRICT

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*Available at Contra Costa Water District, 2411 Bisso Lane, Concord, CA 94520

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List of Acronyms and Abbreviations

AB 32	Assembly Bill 32
AB 1493	Assembly Bill 1493
ABAG	Association of Bay Area Governments
AF	acre-feet
AFY	acre-feet/year
APE	area of potential effects
ARB	Air Resources Board
BAAQMD	Bay Area Air Quality Management District
BC	Before Christ
BCDC	San Francisco Bay Conservation and Development Commission
BMPs	Best Management Practices
BO	Biological Opinion
BRA	Biological Resources Analysis
CAA	Clean Air Act
CAAQS	California Ambient Air Quality Standards
CAP	Clean Air Plan
CCR	California Code of Regulations
CCWD	Contra Costa Water District
CDFG	California Department of Fish and Game
CEC	California Energy Commission
CEQA	California Environmental Quality Act
CFR	Code of Federal regulations
cfs	cubic-feet per second
CGP	Consolidated General Permit
CH_4	methane
cm	centimeter
CNDDB	California Natural Diversity Database
CO	carbon monoxide
CO_2	carbon dioxide
CO_2e	carbon dioxide equivalents
COE	U.S. Army Corps of Engineers
CNDDB	California Natural Diversity Data Base
CNPS	California Native Plant Society
CWA	Clean Water Act
dB	decibels
dBA	A-weighted decibels
Dbh	diameter at breast height
DOC	California Department of Conservation
DMG	Division of Mines and Geology
DPM	diesel particulate matter
DTSC	Department of Toxic Substances Control
DWR	Department of Water Resources
EA	Environmental Assessment
EBMUD	East Bay Municipal Utility District
EBRPD	East Bay Regional Park District
EPA	Environmental Protection Agency

ESA	Endangered Species Act
ft	feet
FEMA	Federal Emergency Management Agency
FONSI	Finding of No Significant Impact
FMMP	Farmland Mapping and Monitoring Program
FWCA	Fish and Wildlife Coordination Act
GHG	greenhouse gases
I	Interstate (Highway)
IPCC	Intergovernmental Panel on Climate Change
IS	Initial Study
ITA	Indian Trust Assets
HFCs	hydro fluorocarbons
JPC	Joint Policy Committee
km	kilometer
lbs.	pounds
LUST	Leaking Underground Storage Tanks
MBTA	Migratory Bird Treaty Act
mg/L	milligrams per liter
MLD	Most Likely Descendant
MND	Mitigated Negative Declaration
MRZ	Mineral Resource Zone
MS4	Municipal Separate Storm Sewer System
MSL	Mean Sea Level
MSM	Mobile Source Measure
MTC	Metropolitan Transportation Commission
$MTCO_2e$	metric tons of carbon dioxide equivalents
N_2O	nitrous oxide
NAAQS	National Ambient Air Quality Standards
NEPA	National Environmental Policy Act
NHPA	National Historic Preservation Act
NIC	Northwest Information Center
NOI	Notice of Intent
NO_x	nitrogen oxides
NPDES	National Pollutant Discharge Elimination System
NRHP	National Register of Historic Places
O&M	operations and maintenance
PFCs	perfluorocarbons
PM	particulate matter
$PM_{2.5}$	particulate matter less than 2.5 microns in diameter
PM_{10}	particulate matter less than 10 microns in diameter
POTW	Publically Owned Treatment Works
ppm	parts per million
PRDs	Permit Registration Documents
Project	Shortcut Pipeline Improvement Project
PVC	polyvinyl chloride
O_3	ozone
RCEM	Road Construction Emission Model
Reclamation	U.S. Dept. of the Interior, Bureau of Reclamation
Recovery Act	American Recovery and Reinvestment Act of 2009
ROG	reactive organic gases
ROW	right-of-way

RWQCB	Regional Water Quality Control Board
SCADA	Supervisory Control and Data Acquisition
SCP	Stormwater Control Plan
SCPL	Shortcut Pipeline
SF_6	sulfur hexafluoride
SFBA	San Francisco Bay Area
SFBAAB	San Francisco Bay Area Air Basin
SFRWQCB	San Francisco Regional Water Quality Control Board
SHPO	State Historic Preservation Officer
SIP	State Implementation Plan
SLIC	Spills, Leaks Investigation and Cleanup
SMHM	salt-marsh harvest mouse
State	State of California
SWPPP	Stormwater Pollution Prevention Plan
TDS	total dissolved solids
g/m^3	micrograms per cubic meter
U.S. (or US)	United States
USC	United States Code
USFWS	U.S. Fish and Wildlife Services
USGS	U.S. Geological Society
VOCs	volatile organic compounds
WDA	Waste Discharge Authorization
WMU	Waste Management Unit
WPT	western pond turtle

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Section 1 Introduction

This Initial Study (IS) has been prepared by the Contra Costa Water District (District) as the lead state agency to satisfy the requirements of the California Environmental Quality Act (CEQA). The Shortcut Pipeline (SCPL) is owned by the United States Dept. of the Interior, Bureau of Reclamation (Reclamation) and a National Environmental Policy Act (NEPA) document will be prepared at a future date. The District has included in this document all required CEQA information and has also included information that will support Reclamation's NEPA document (EA/FONSI).[1]

1.1 Background / Summary Project Description

The Martinez Reservoir is the terminus of the Contra Costa Canal system, and the SCPL is used to balance flows and allow reliable service to all CCWD customers. The SCPL is the main source of water supply for the City of Martinez, California, the Foster Wheeler Power Plant within the Tesoro Refinery, as well as the Shell Oil Martinez refinery. It is approximately 5½ miles (28,000 feet) long and located generally in north-central Contra Costa County within a 40-foot, non-exclusive easement that passes through several private and publicly owned property parcels (see **Figure 1**).

1.1.1 Summary SCPL Description

The SCPL is a cement-mortar-lined and coated steel water supply pipeline that was built in 1972 and conveys untreated water from the Contra Costa Canal to the Martinez Reservoir. **Figure 1** shows an aerial photo of the project area within the SCPL, shown in blue. On the eastern end, the SCPL connects to the Contra Costa Canal. The SCPL proceeds on a generally westerly course to its terminus with Martinez Reservoir. Prior to construction of the SCPL, deliveries to Martinez Reservoir followed the portion of the Contra Costa Canal known as the Loop Canal as it proceeded south from Clyde, then looped through the cities of Concord, Walnut Creek, Pleasant Hill and Martinez to Martinez Reservoir. The SCPL consists of 42-, 48-, and 60-inch-diameter pipe segments. There are pipeline appurtenances associated with the SCPL that include numerous valves that allow for sealing, draining and refilling of the pipe, settlement monitors to allow for detection of SCPL settling, and a cathodic protection system to prevent corrosion. Access to the pipeline and infrastructure is gained through various routes.

The District has identified critical repairs and improvements needed to ensure reliable long-term water supply using the SCPL. For example, there are 5 valves that are currently not readily accessible for maintenance or repair due to lack of usable roads at Sites 5, 7, and 10 (see **Table 1** and **Figure 6**). The SCPL Improvement Project (Project) includes unpaved gravel access road construction and infrastructure rehabilitation activities, followed by permanent and long-term ongoing operation and maintenance (O&M) activities. These O&M activities, currently constrained due to limited access conditions, will be facilitated by the planned access road construction. **Table 1** summarizes information about the Project sites and **Table 2** summarizes the work activities at each site in chronological order. A limited number of Phase 1 repairs were completed in January 2011 within portions of the pipeline easement within the Shell and Tesoro Oil Refineries. Phase 1 work was covered under a Categorical Exclusion Checklist (CEC) dated April 1, 2010.

[1] Environmental Assessment/Finding of No Significant Impact.

The site figures in **Appendix B** display visual information about Sites 1 through 10, including valve and access road location, as well as detailed information on biological and hazardous material issues. It is suggested that the reader of this report keep these maps in foldout form for reference while reading the report.

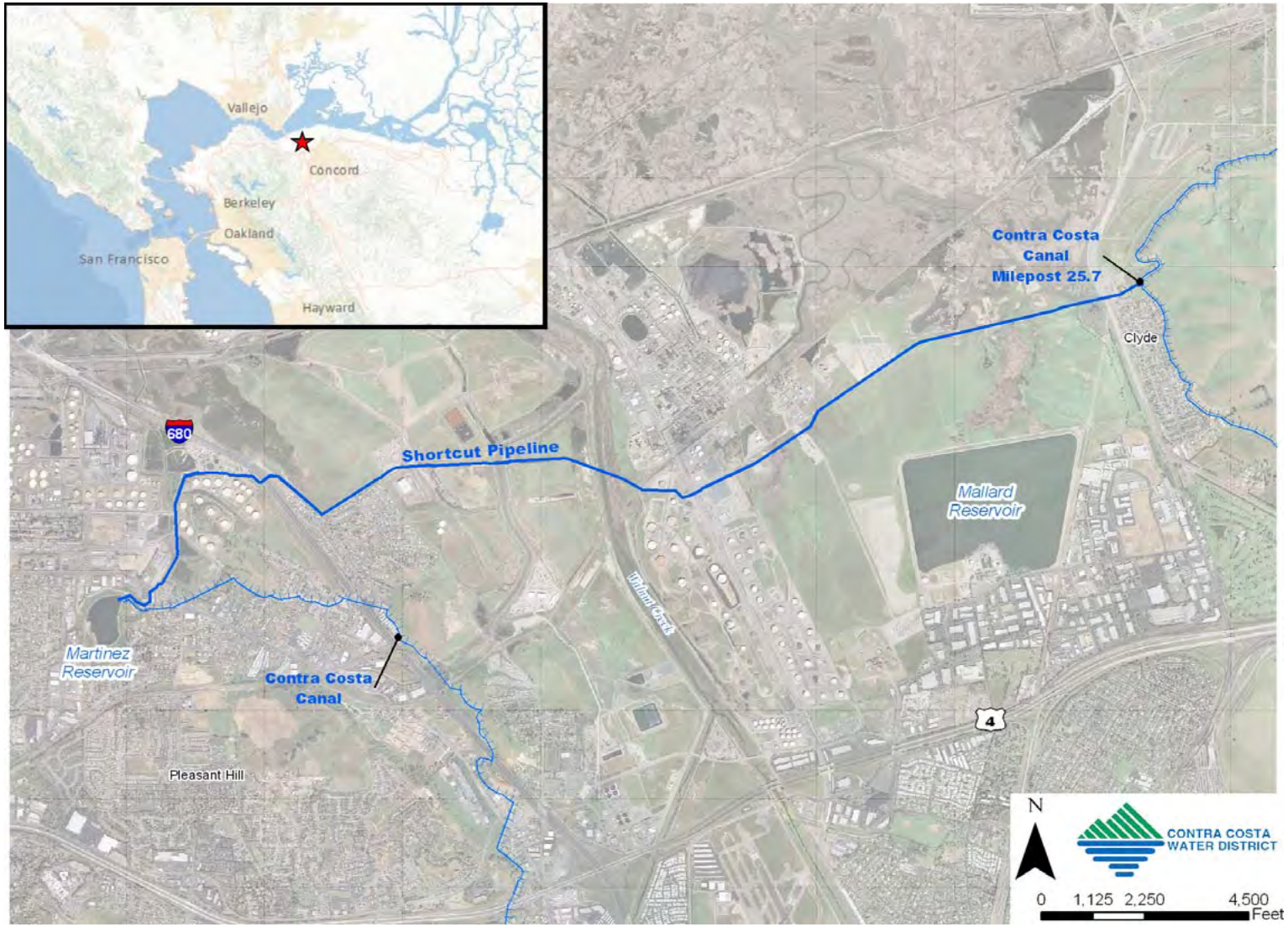

Figure 1 - Location and Site Vicinity Map

Source: CCWD

Table 1 Summary of Information About Each Site					
Work Site	**Site Description**	**Proposed Improvements**	**Land Use**	**Habitats and Wetlands**	**Hazardous Materials**
Site 1	Shell Refinery	Settlement Monitors	Heavy Industrial	None	Potential
Site 2	Tree Service Company	Blow-off Valve	Heavy Industrial	Adjacent to McNabney Wetlands	Minimal
Site 3	Martinez Gun Club	Settlement Monitors, Blow-off Valve	Heavy Industrial	Grasslands	Minimal
Site 4	IT Ponds	Air Valve, Settlement Monitors and Access Road	Heavy Industrial	Wetlands	Medium potential
Site 5	Foster Wheeler Power Plant	Access Road, Blow-off Valve and new Butterfly Valve, Settlement Monitor	Heavy Industrial	Grasslands and wetlands	Minimal
Site 6	Tesoro Refinery	Blow-off, Air and Butterfly Valves, Access Road, Settlement Monitors	Heavy Industrial	None	Potential (WMU4)[1]
Site 7	Tesoro Refinery	Blow-off Valve and Access Road	Heavy Industrial	Grasslands and wetlands	Minimal
Site 8	Tesoro Refinery	Air Valve,	Heavy Industrial	Grasslands	Minimal
Site 9	Tesoro-end of Monsanto Way	Air Valve, Blow-off Valve, Settlement Monitors	Heavy Industrial	Grasslands	Minimal
Site 10	Concord Naval Weapons Station	Blow-off, Air and Butterfly Valves, new Access Road, Settlement Monitors	Heavy Industrial	Grasslands and wetlands	Minimal

[1]Waste Management Unit 4 within the Tesoro Golden Eagle Refinery.

Table 2 List of Activities and Estimated Time Frames for the SCPL Improvement Project		
Activity	**Subactivity**	**Construction or Maintenance Timeframe**
Pipeline Assessment Phase **Complete**	1. Inspect pipeline valves. 2. Third–party access agreements have been obtained as required.	Completed Completed
Phase 1 Valve Repair & Refurbishment Work began in June 2010 and was completed in early 2011. Sites 1, 3, and adjacent to Site 5.	1. Refurbish 3 existing Air Valves and 1 Blow–off Valve. 2. Replace 3 Butterfly Valves and construct 4 new Air Valves. 3. Construct 500–foot gravel haul road (Tesoro property). 4. At Contra Costa Canal construct 18–inch air vent adjacent to SCPL slide gate. 5. Maintain valves that have been repaired or newly installed. Maintain new access road on Tesoro property.	Completed Completed Completed

Table 2 List of Activities and Estimated Time Frames for the SCPL Improvement Project		
Activity	**Subactivity**	**Construction or Maintenance Timeframe**
Phase 2A	1. Construct 5 new gravel at-grade access roads. a. Approximately 1,900 feet along the easternmost segment of the pipeline (to access 1 Butterfly Valve, 1 existing Air Valve, 1 new Air Valve and 2 Blow-off Valves). Site 10. b. 650 feet of access road at Site 7 within the Tesoro Refinery. Site 7 is east of WMU4; the road provides access to a Blow-off Valve. c. Approximately 450 feet of road west of the Foster Wheeler power plant (to access a Blow-off Valve adjacent to Pacheco Slough and for installation of a new Butterfly Valve). Site 5. d. Approximately 2,000 feet adjacent to IT Corporation property (the area where the pipeline failed during the 1989 Loma Prieta earthquake) with access to 1 Air Valve and numerous settlement monitors. Site 4. e. Construction of minor gravel, paved or concrete access to 2 Air Valves, both along Monsanto Way east of WMU4 site (Sites 8 and 9). Regraveling of approximately 480 feet of existing Monsanto Way road. Site 9.	Summer/Fall (2012 and 2013)
	2. Refurbish 3 existing Air Valves (Sites 4, 8, 9, and 10) and 6 Blow–off Valves (Sites 2, 5, 7, 9, and 2 valves at 10). 3. Replace 1 Butterfly Valve and construct 1 new Air Valve. Site 10. 4. Construct new Butterfly Valve. Site 5. 5. Install or replace up to 40 settlement monitors.	Following road construction in 2012 and 2013 (Subactivities 2-5)

Table 2 List of Activities and Estimated Time Frames for the SCPL Improvement Project		
Activity	**Subactivity**	**Construction or Maintenance Timeframe**
Phase 2B	1. Work within WMU4 site on Tesoro Golden Eagle Refinery property. 130 feet of access road construction, refurbish Blow-off Valve, replace Air Valve, replace Butterfly Valve construct new Air Valve, install settlement monitors. Site 6. 2. Construct levee road turnout and refurbish 1 Blow-off Valve east of the Martinez Gun Club (Site 3).	Pending discussions with the Avon Remediation team (ART)[2] and the SFRWQCB.
Phase 3	1. Inspect pipeline in the area where prior break occurred in 1989. Site 4. 2. Based on the inspection, if necessary, repair pipeline section either through slip lining existing pipeline, spot repairs with excavation or via trenching and installation of replacement pipeline.	2013 2014
Routine Operations and Maintenance	1. Access pipeline using access roads. Test valves as required. Inspect and repair cathodic protection system as needed. Inspect settlement monitors. 2. Perform maintenance as required (including weed abatement) while minimizing and avoiding impacts to sensitive resources.	Starting in 2012 Ongoing once repairs are made

1.1.2 Purpose and Need/Project Objectives

The purpose of the proposed Project is to perform repairs to the existing pipeline and associated valves, construct new access roads to some of the inaccessible portions of the SCPL, install settlement monitors, and improve operational flexibility so that maintenance and repairs can be performed with reduced environmental effects.

New and replacement valves are needed on the SCPL to allow for greater isolation of pipeline segments, so that only partial sections need to be drained to perform repairs, thereby reducing the amount of discharge/flooding and the amount of water wasted. Valve work also allows portions of the pipeline to remain in service while repairs are being made to the isolated sections. The 1989 Loma Prieta earthquake illustrated this need when it resulted in the rupture of a section of the SCPL. New roads are needed to provide access to the SCPL to facilitate the performance of critical maintenance activities, while reducing the impact of those activities. The pipeline crosses sensitive wetland habitat near Suisun Bay. Without access roads to valves located in these areas, trucks and equipment could damage the sensitive habitat and some sections of the SCPL would remain inaccessible for maintenance activities during periods when these areas are wet.

[2] ART includes Tesoro and Teseco Downstream Properties, a subsidiary of Chevron Environmental Management Company.

The primary objective of the proposed Project is to ensure reliable long-term untreated water supply to our customers, including the City of Martinez, Foster Wheeler Power Plant within the Tesoro Refinery, and the Shell Oil Refinery using the SCPL. Maintaining the SCPL in good operational condition is critical to this objective because the alternative means for conveying water to the Martinez Reservoir—the Loop Canal—is taken out of service for 3 months each year for maintenance, such as vegetation clearance from the canal channel. In addition, the Loop Canal is a less efficient and less reliable means of conveying water to Martinez Reservoir, and at times is incapable of fully meeting the peak water demand of the City and the Shell Oil Refinery. Finally, Martinez Reservoir is the terminus of the Contra Costa Canal system and is used to balance flows and allow reliable service to all of CCWD's customers. The SCPL is a necessary element to provide that reliable service. The SCPL also facilitates better management of the Loop Canal.

1.1.3 Scope/Project Location and Setting

The SCPL is located in north-central Contra Costa County, approximately 1.5 miles north of State Highway 4 and about 1.5 miles south of the San Joaquin/Sacramento Delta. The western portion of the alignment crosses Interstate 680 (I–680). The alignment passes through undeveloped open space (primarily grassland and marshland), 2 refineries, and industrial development; it also passes adjacent to two residential neighborhoods. The SCPL alignment traverses a number of property boundaries as shown in **Figure 2**. Existing conditions along the SCPL alignment are depicted in **Figures 3 and 4**.

The eastern end of the SPCL begins at the Contra Costa Canal, at the northern edge of the unincorporated community of Clyde, approximately 950 feet east of the Port Chicago Highway and approximately 3,500 feet northeast of Mallard Reservoir. The alignment runs adjacent to an existing residential neighborhood within Clyde, then crosses under the Port Chicago Highway and begins traversing open fields containing salt marsh and low-lying grasslands. Approximately 4,000 feet west of the Port Chicago Highway, the alignment turns to the southwest and follows Monsanto Way, a private road within the Tesoro Golden Eagle Refinery property.

Continuing in a southwesterly direction, the SCPL crosses developed Tesoro refinery property, then crosses under Walnut Creek, continues west for another 2,000 feet across a flat, vacant field, then crosses under Pacheco Creek. The SCPL continues west for approximately 1,800 feet across open ground, passing adjacent to the Martinez Gun Club property. It follows Arthur Road in a southwesterly direction, passing between a residential subdivision and open space property owned by the East Bay Regional Park District. After jogging under Interstate I-680, the alignment passes through the Shell Oil Martinez Refinery, crosses Pacheco Boulevard, passing a commercial property, then veers east and terminates at the Martinez Reservoir.

The topography along the majority of the alignment is relatively flat, and elevations along most of the pipeline range from just below sea level to about 20 feet above sea level. Near the western end, the SCPL winds through several hills and terminates at the Martinez Reservoir at an elevation of 100 feet.

1.1.4 Waste Handling Units Along SCPL Alignment

Waste handling was performed in the past on 4 facilities located near the SCPL under separate U.S. EPA facility identification numbers: CAD000092619 – labeled "Former IT Corporation Waste Pond (see Site Map 4, Appendix B); CAD000094771 – labeled "IT Group" and "Former

IT Corporation Waste Pond" (see Site Maps 3 and 4, Appendix B); CAD089680250 – labeled "Former IT Corporation Waste Pond" (see Site Map 4); and CAD041835695 – labeled "IT Group" (see Site Map 3, Appendix B). All 4 facilities, used before 1989 for waste treatment and liquid waste impoundments, in the 1990s underwent closure processes and today are in post-closure monitoring phases. In addition, Site 6 is located within a known hazardous waste site (WMU4) and of immediate concern is the October 2010 San Francisco Bay RWQCB conditional approval of the Closure and Post-Closure Maintenance Plan, WMU4, Golden Eagle Refinery in Martinez, California.[3] The Avon Remediation Team (ART), comprised of Tesoro and Chevron-Environmental Management Company for Texaco Downstream Properties, Inc., is implementing the planned closure at WMU4. Communication has been established with ART regarding the planned closure of the waste site and its relationship to the existing SCPL.

Remediation of all of the waste sites with respect to the SCPL is discussed in **Section 3.9, Hazards and Hazardous Materials**. Locations of key waste sites are depicted in **Figure 5** and on the individual site figures in **Appendix B.**

1.2 Project Sites and Activities

Figure 6 presents the proposed 10 sites for Phase 2. There are 4 possible Project work activities at each site:

- Check for hazardous materials at each site (see **Section 3.9 Hazards and Hazardous Materials** for procedures and details)
- Gravel access road construction
- Valve replacement or installation
- Settlement monitor replacement or installation

[3] SCPL Hazardous Materials Review, Environmental Science, 2011.

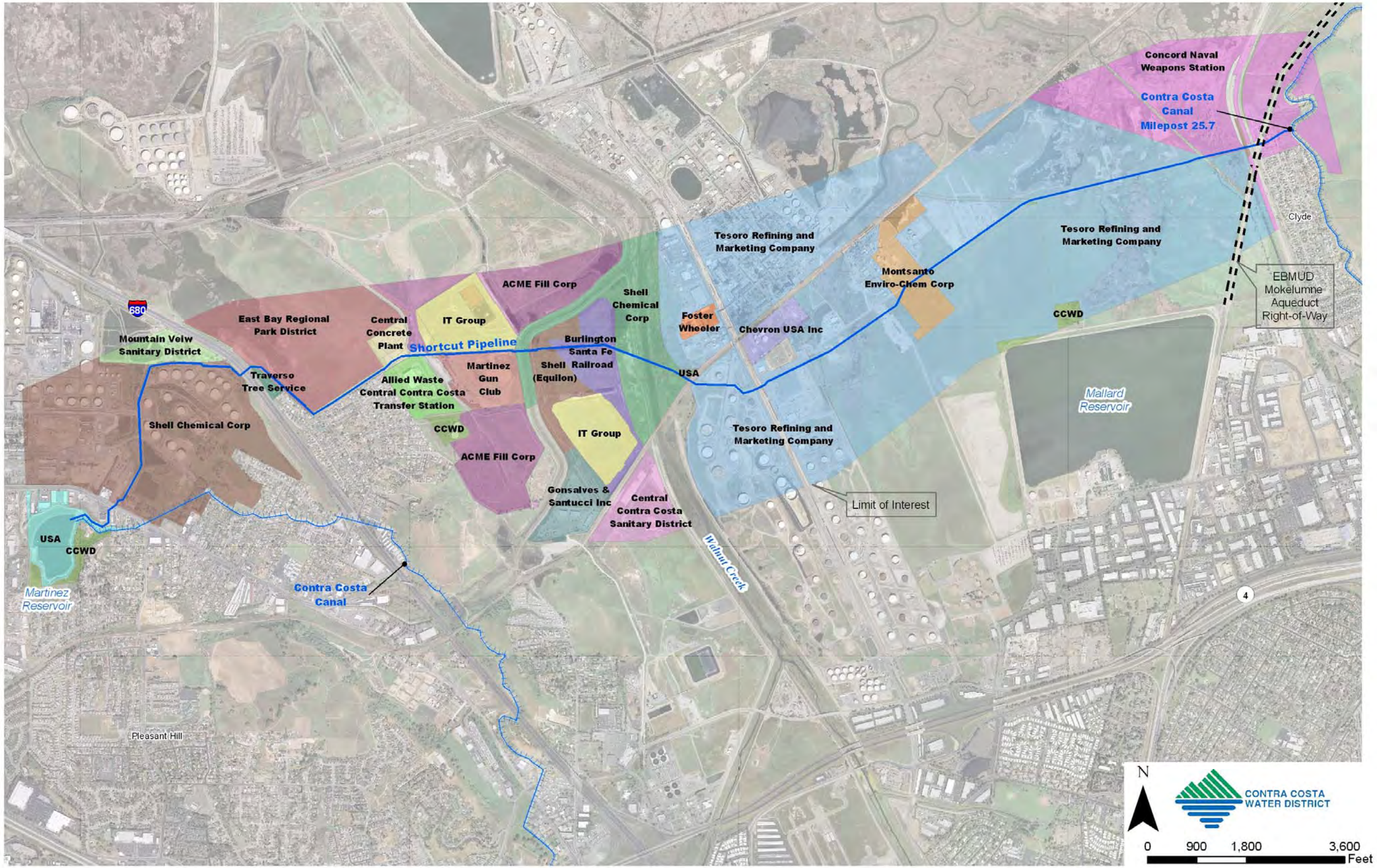

Figure 2 - Property Boundaries along Pipeline Route

Source: CCWD

Wetland marsh habitat in the vicinity of the SCPL, looking west towards Tesoro Refinery. Existing Blow-off Valve located north (right) of the fence line.

Figure 3 - Existing Conditions Along SCPL Alignment (Site 10) **Source: LSA**

Looking east toward Clyde, Pickleweed habitat on the SCPL Right of Way at Site 10.

Existing Blow-off Valve located alongside Montsanto Way, looking west toward Tesoro Refinery at Site 9.

Figure 4 - Existing Conditions Along SCPL Alignment (Sites 9 & 10) **Source: LSA**

1.2.1 Planned Activities by Site

The planned activities at each site for Phase 2 are summarized in **Table 3**. Phase 3 work activities would involve inspection and potential repair of the pipeline in Site 4. The individual activities are described in more detail in the following sections. Maps for sites 1-10 in **Appendix B** depict the specific activities that will occur at each site. The right-of-way (ROW) and construction easements are also shown on the maps, as well as the identified jurisdictional wetlands and Waters of the U.S. discussed in **Section 3**.

Table 3
Phase 2 Planned Activities

Site No.	Access Roads	Butterfly Valves	Blow-off Valves	Air Valves	Settlement Monitors[3]	Tentative Schedule
1	-	-	-	-	1	Fall 2012
2	-	-	1	-	-	Fall 2012
3	-	-	1	-	2	Fall 2013
4	2,000 ft	-	-	1	-	Fall 2012
5	450 ft	1	1	-	1	Fall 2012
6	130 ft	1	1	2	1	Fall 2013
7	650 ft	-	1	-	1	Fall 2012
8	-	-	-	1	-	Fall 2012
9	480 ft[1]	-	1	1	1	Fall 2012
10	1,900 ft[2]	1	2	2	4	Fall 2012
Total	**5,610 ft**	**3**	**8**	**7**	**11**	Fall 2012-Fall 2013

[1]480 feet of existing road will be re-graveled.
[2]Would require up to 5 culvert crossings.
[3] Eleven proposed settlement monitors in addition to 11 existing monitors and 18 optional locations, as shown in Figure 12.

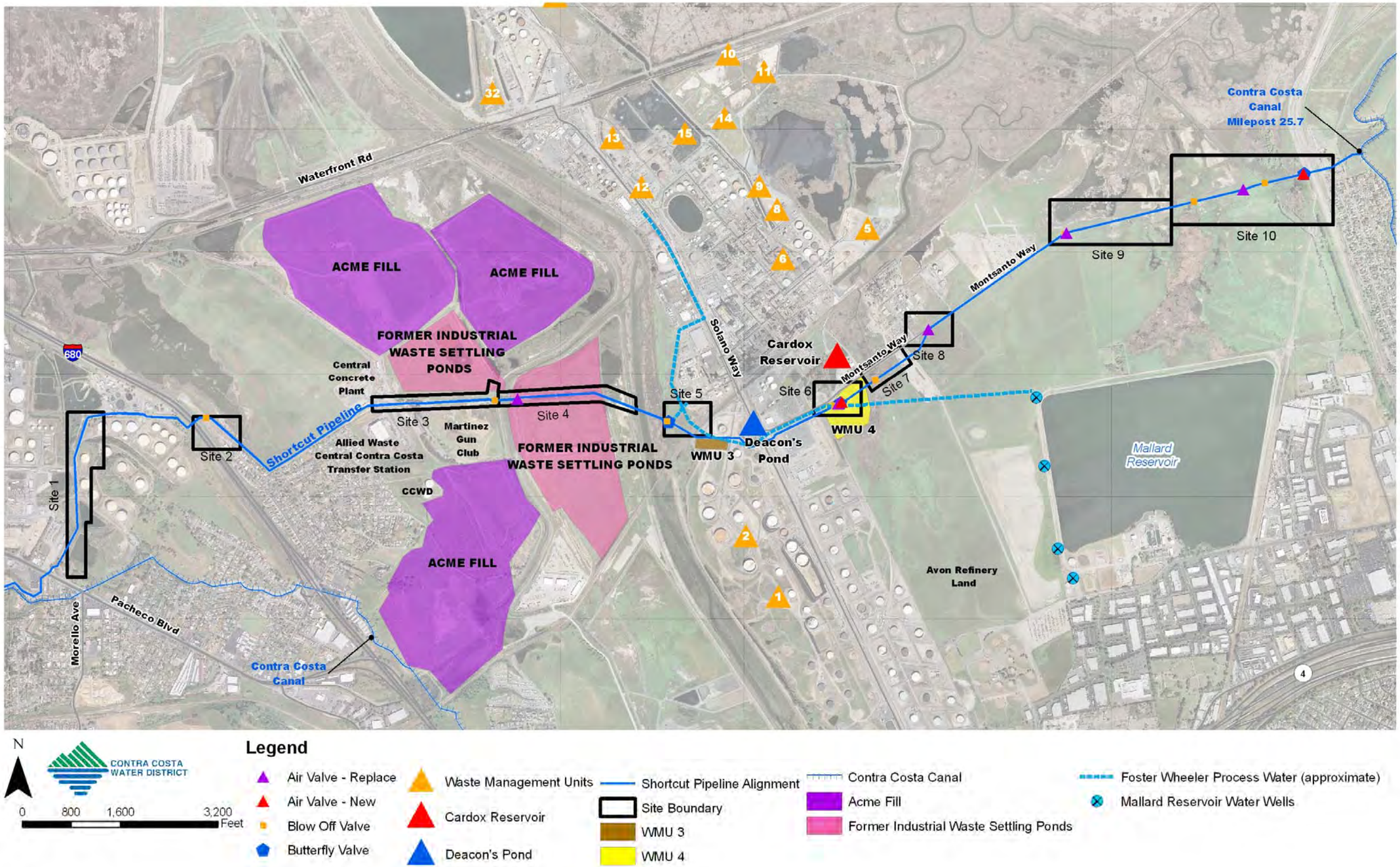

Figure 5 - Hazardous Waste Sites in the SCPL Vicinity

Source: CCWD

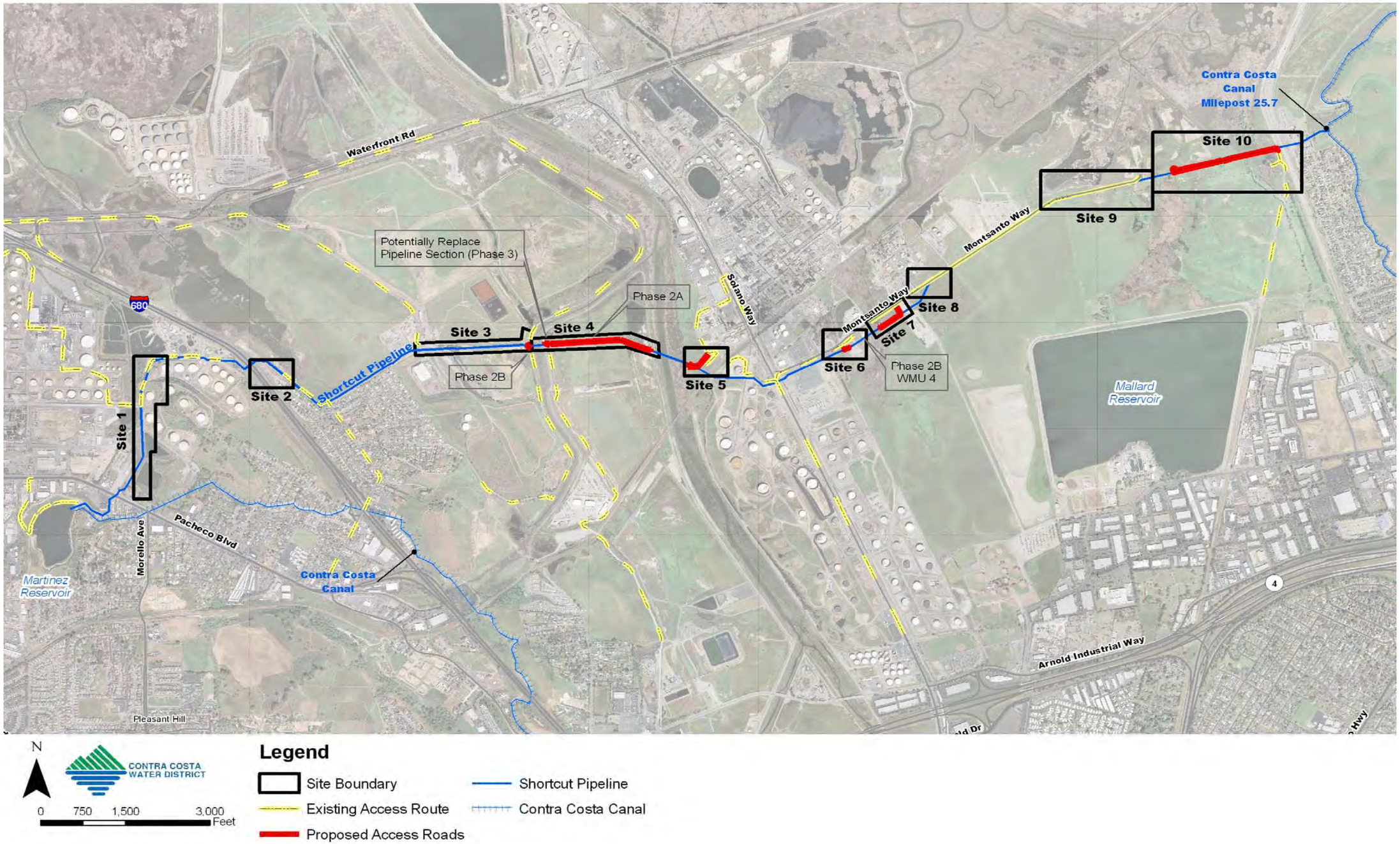

Figure 6 - Work Sites and Existing & Proposed Access Roads

Source: CCWD

1.2.2 Hazardous Materials

Property owners and the SFRWQCB will be contacted regarding hazardous materials. If required, testing near the Sites will be carried out. If hazards are present, a plan for treatment and disposal will be prepared (see details in **Section 3.9 Hazards and Hazardous Materials**). Pre-construction testing will be performed in locations where possible hazardous wastes may be present in the vicinity of the Project.

1.2.3 Access Road Construction

Gravel access road construction is a key component of the Project. SCPL maintenance and operation activities require that District maintenance staff have the ability to access valves, cathodic protection equipment, settlement monitoring stations, and other stations along the pipeline alignment. Gravel access roads are required to support safe maintenance of the SCPL. New gravel access roads are required to access certain facilities. Once access is established, ongoing maintenance activities include mowing, hand weed abatement, and herbicide applications as well as routine operations to open and close all of the valves on the SCPL. Descriptions of the required maintenance activities are provided in the following section. District staff will work with Reclamation to seek timely reviews for these maintenance activities from United States Fish and Wildlife Service (USFWS) and the U.S. Army Corps of Engineers (COE) as required.

Need for Improved SCPL Access

Access to the SCPL for repairs and maintenance is currently constrained; in certain locations there are no existing access roads, and in other locations the existing access routes pass close to environmentally sensitive areas or are inundated during the rainy season. There are portions of the SCPL right-of-way outside of sensitive environmental areas and these areas have existing vehicle access, in most cases on paved roadways.

Repair and construction of new access roads along the SCPL alignment will provide for routine maintenance in areas that are in close proximity to wetlands and sensitive habitats for special-status plants and/or wildlife species. Access road construction is intended to minimize the potential for impacts to sensitive resources and ensure safe maintenance and repair of the pipeline. It will be necessary to operate all of the valves along the SCPL at least once per year.

Description of Planned Access Roads

The District has obtained a long-term license from Contra Costa County to utilize roads to access the pipeline at Sites 3 and 4. The District is working with the East Bay Municipal Utility District (EBMUD) to obtain any related permits or licenses to use their maintenance road along the Mokelumne Aqueduct ROW to access Site 10.

The Project would require construction of 5 primary segments of new access roads; the locations are shown in **Figure 6**. Additional minor road segments would be constructed at some locations to provide connections to existing roads. Approximately 1 mile (5,130 feet) of new access roads would be constructed, and an additional 480 feet of existing road located in Site 9 would be re-graveled. The roads would be 12 feet wide and surfaced with compacted gravel. An example of an access road completed in Phase 1 is shown in **Figure 7**. All new access road construction would take place within the existing Reclamation ROW with the exception of approximately 160

feet of road at Site 7, and approximately 115 feet of road in 2 segments at Site 10. At Site 7, the access road would extend north beyond the ROW to connect to Monsanto Way on the Tesoro Golden Eagle Refinery property. At Site 10, 100 feet of road would extend north outside the ROW to form a turn-around at the western terminus, and 15 feet would be constructed to the south of the ROW to meet an existing access road on Tesoro Refinery property.

Road Construction Methods

To construct the new access roads, a dozer would clear, grub (clear of roots and stumps), and scarify (remove any existing pavement) the ground, then an excavator would over-excavate the roadbed to a depth of 1 to 2 feet below the natural ground surface. Debris and excess soil would be off-hauled for disposal at an off-site location. The roadbed would be uniformly graded and crowned for positive drainage away from the road. A non-woven geotextile fabric would be laid, then the roadbed would be backfilled with soil, crushed rock, and compacted with a vibratory compactor. The roadbed—consisting of imported Caltrans Type II aggregate base—would be built up to a height of 4 to 6 inches, including a geofabric layer for road strength and stability. In some locations, the road would be at grade rather than built up. The roadbed would be finished with gravel compacted by a two-roller compactor.

Culverts under the proposed access roads would be required at some locations to maintain existing hydrology, including storm drainage and encroachment from particularly high tides (higher high water) or 100-year flood events. It is estimated that approximately up to 5 culverts would be needed at Site 10. However, final locations and design of the culverts will determined at the time of construction. It is anticipated that the culverts will be part of surface road construction at this location.

Construction of the access roads is planned to commence in fall 2012 and would require 4 to 8 weeks to complete. Some roads may be constructed concurrently with the valve work (subject to environmental constraints). Approximately 8,190 cubic yards (cy) of fill material (primarily aggregate) would be imported, resulting in approximately 546 round trip truck trips to the sites, or an average of about 84 one-way trips per work day, or 42 round trips over the construction period. Only 10 valves will require excavation of soils. It is estimated that no more than 300 cubic yards would be exported from the site if all of the excavated materials were removed. Exported spoils would be less than imported fill. Because it is unlikely that trucks exporting excavated spoils off the site during valve repair and replacement would involve the same trucks importing fill to the site, an additional 20 one-way trips and 10 round trips would be added, resulting in a total of approximately 100 one-way trips and 50 round trips per workday. Any new hazardous materials found within the ROW are expected to be disposed of at approved waste disposal sites. As described in the hazardous materials **Section 3.9**, the District will be contacting existing property owners and seeking to dispose of any hazards at the property owner's approved disposal site.

Proposed access roads in sensitive areas would require the permanent filling of 1.23 acres of wetlands and other Waters of the United States (plus 6.49 acres on a temporary basis), requiring permits from the COE and other agencies. Additional information is provided in Section 1.3, **Required Permits and Approvals**, and in Section 3.4, **Biological Resources**. The Project will include onsite restoration of temporary impacts. Wetland mitigation may be provided via payment to the East Contra Costa County Habitat Conservancy and habitat mitigation may be paid via in lieu fees.

Example of new access road (on Tesoro Golden Eagle Refinery property adjacent to Waste Management Unit 3) completed during Phase 1 of the SCPL Improvement Project (Southeast of Site 5).

Figure 7 - Photo of Recently Completed Access Road (Phase 1, Completed 2010/2011)

Source: CCWD

1.2.4 Valve Replacement and Installation

There are 3 types of valves on the SCPL: Butterfly Valves, Blow-off Valves and combination Air Valves. Phase 1 replacement of valves began in June 2010 and was completed in January 2011 at locations on the SCPL where existing paved access roads already provided impact-free access to the valve locations. Phase 1 construction was covered under a previous Categorical Exclusion pursuant to NEPA.[4] A total of 3 existing Air Valves and 1 Blow–off Valve were refurbished as part of the approved Phase 1 work. Phase 1 also included replacement of 3 existing Butterfly Valves and construction of 4 new Air Valves.

Valve refurbishment and replacement would continue during Phase 2 of the Proposed Project, which would commence in fall 2012 following completion of the new access roads. Phase 2A is expected to be completed by January 2013. Phase 2B construction is estimated to be completed by January 2014 but still require Contra Costa County authorization of work on its levee at Site 3 and a satisfactory resolution of any concerns at Site 6 (WMU4). In total, 2 Butterfly Valves, 8 Blow-off Valves, and 5 Air Valves would be replaced and 2 new Air Valves and 1 new Butterfly Valve would be installed. The locations of all new valves and valves to be replaced as part of the Project are shown in **Figure 8**.

Butterfly Valves

Butterfly Valves are located at strategic locations along the SCPL to regulate or shut off water flow to the pipeline or isolate water within certain sections of pipe. There are a total of 5, 48-inch Butterfly Valves on the SCPL. Encased in cast-iron with a flange that wraps around the pipeline, the valve consists of a metal disc mounted on a rod or pipe bisecting the pipe laterally. When a valve nut is turned, the disc rotates to an open or closed position. In the closed position, the valve disc rests against a flat ridge plate inside the pipeline, sealing off the flow of water. In the open position, the valve disc rotates to a position parallel with the sides of the pipe, and water flows on either side of the valve. A valve operator attached to the side of the Butterfly Valve houses the gear mechanism that rotates the disc inside the pipeline. A valve stem extension housed in 8-inch PVC piping extends upward above the ground surface, where the operating valve nut is enclosed in a housing and can be operated by authorized District personnel. The Project will replace 2 Butterfly Valves at Site 10 and Site 6 and add 1 new Butterfly Valve at Site 5. The new Butterfly Valve at Site 5 will support maintenance of the intertie between the SCPL and the Foster Wheeler Power Plant. A photo of a typical ground-level housing for a Butterfly Valve control is shown in **Figure 9.**

Blow-off Valves

Blow-off Valves are located at intervals along low points on the SCPL to allow for drainage of isolated pipeline segments for purposes of repairs or maintenance. There are a total of 9, 6-inch Blow-off Valves on the SCPL. These valves therefore are instrumental in minimizing the amount of water that must be discharged from the pipeline prior to making a repair. The steel valves are housed in a steel-reinforced concrete manhole and are connected to the SCPL with an outlet nozzle and reinforcement collar. When the gate valve is opened, water drains from the SCPL into a discharge pipe that drains to an appropriate downstream discharge location. As the Blow-off Valves are replaced, some discharge pipes will be abandoned in place and new flexible pipes used for discharge. The flexible pipe will allow water to drain to a nearby discharge

[4] Contra Costa Water District, *Categorical Exclusion Checklist: CCWD Shortcut Pipeline Valve Rehabilitation–Phase 1*, January 21, 2010.

location or if necessary to tanks that could be trucked away. A photo of a typical manhole for a Blow-off Valve is shown in **Figure 10.**

Air Valves

Air Valves allow air to be drawn into the pipe and are necessary to avoid damage that could result from negative pressure (vacuum) developing during dewatering or failure of the pipe, such as might occur during a seismic event. The Air Valves also function to release excess air when the pipe is filled. There are currently a total of 8, 8-inch Air Valves on the SCPL. The Air Valves are located at high points (elevations) on the SCPL; they are located above ground, mounted vertically on a concrete pad and protected in a locked, vented enclosure. They are laterally offset from the pipeline by approximately 8-feet, connected to the SCPL by steel riser pipe and appropriate fittings. A photo of a typical Air Valve housing and an example of a newly installed Air Valve is shown in **Figure 11.**

In addition to replacing 5 existing Air Valves, the Project would install 2 new Air Valves to provide increased operational flexibility in taking limited pipe segments out of service for maintenance. One would be located at Site 10, just downstream of the first Butterfly Valve at the eastern end of the SCPL. The second would be located at Site 6, within the Tesoro Golden Eagle Refinery. These new locations will ensure there is an Air Valve on each side of a Butterfly Valve.

Valve Rehabilitation and Installation Procedures

Replacement or new installation of Butterfly and Air Valves requires excavation around the pipeline in an area measuring approximately 10 feet wide by 10 feet long, with depths up to 10 feet. Replacement of the Blow-off Valves would not require any excavation; the valves would be accessed through the existing manhole covers, with workers climbing down to the valves on ladders mounted in the manholes.

Prior to excavation along the pipeline in areas of suspected or known contamination, the District would determine if the valve location has potential for hazardous waste. The District would follow the procedures for hazardous waste set forth in Section 3.9. For excavations around the pipeline, crushed stone foundation and bedding material would be used to stabilize the pipe. A minimum 12-inch thick layer of foundation material would be placed at the bottom of the excavation and around the new manhole structures. Additional thickness of foundation material would be placed where native soils in the excavation bottom are unstable. Foundation material would consist of clean, natural 1.5-inch crushed (i.e. angular) rock.

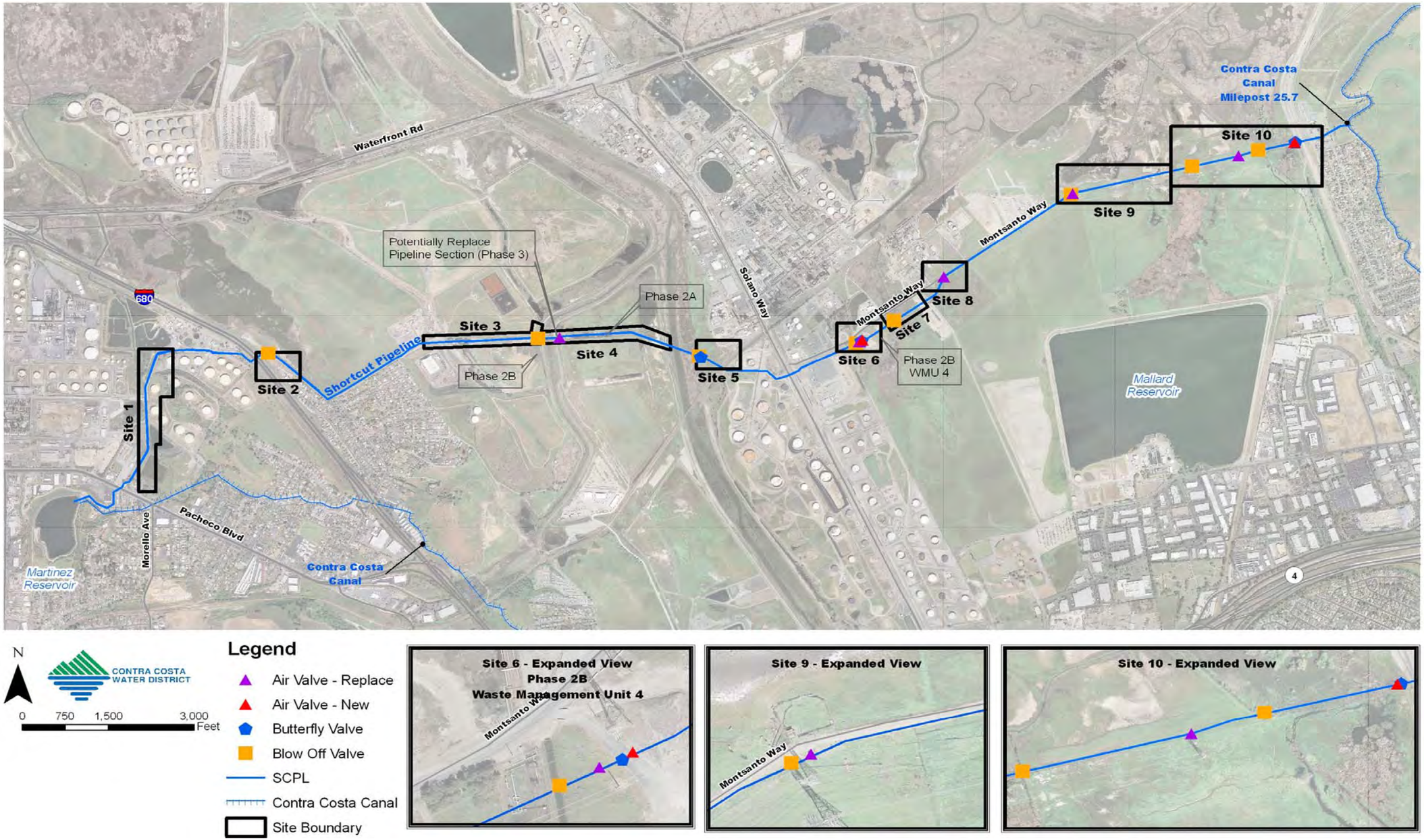

Figure 8 - Proposed New & Replacement Valve Locations Source: CCWD

Example of a Butterfly can lid (in foreground) with Air Valve in background located at Site 6, within Tesoro's Waste Management Unit 4 (WMU4).

Figure 9 - Typical Butterfly Valve Can Lid (in blue) **Source: CCWD**

New blow-off valve installed during Phase 1 of the SCPL Improvement Project.

Figure 10 - Typical Blow-off Valve **Source: CCWD**

During replacement or installation of new valves, the SCPL will be shut down and segments of the SCPL would be drained to provide necessary access. Raw (untreated) water would be discharged via the Blow-off Valves, with the water drained down to an intermediate water level, below the elevation of the particular valve to be replaced. There are 5 major elevation peaks along the SCPL, each separated by butterfly isolation valves (see **Figure 9**). If the Butterfly Valves are operable, the Butterfly Valves would be closed so that only SCPL segments where the work is occurring would be drained. Draining would require disposal of untreated water from several discharge points. Possible discharge locations include nearby Walnut, Pacheco, or Seal creeks or other storm water conveyance structures. The U.S. Environmental Protection Agency (EPA) has determined that the discharge of untreated water to waters of the U.S. (such as the aforementioned creeks) is exempt from the need to obtain a discharge permit pursuant to the National Pollutant Discharge Elimination System (NPDES) established under the Federal Clean Water Act of 1972.[5] Groundwater may be encountered during excavation to install some valves and settlement monitors, and dewatering of the excavations would be required. Groundwater within excavated valve sites will either be applied to land or if necessary pumped into holding tanks and hauled off site.

Excavated areas near valves may require minor dewatering. In most cases groundwater quantities are expected to be minor. The District would dispose of any groundwater containing hazardous materials at an appropriate location for such materials. If groundwater levels are greater than anticipated, construction of temporary detention basins may be employed at some locations for groundwater disposal from dewatering sites.

1.2.5 Settlement Monitors Installation and Replacement

The proposed Project also includes the installation of settlement monitors at several locations along the SCPL to monitor settlement that may occur with natural ground subsidence or as a result of seismic activity (see **Figure 12**). An example of a settlement monitor is shown in **Figure 13.** Monitoring the pipeline for settlement is critical to preventing potential damage of the pipe.

Although the exact number and locations are still being determined, up to 40 settlement monitors would be installed along the SCPL. New settlement monitors will be installed where valve repair or replacement will occur or where new valve installation is planned. New settlement monitors will be sited to avoid disturbance of sensitive habitat. Installation of new monitors would require excavation of an area about 10 feet wide and 10 feet long, down to the depth of the top of the pipe.

[5] California Regional Water Quality Control Board, Central Valley Region, 2009, “Notice of Termination; General Waste Discharge Requirements Order No. 5-00-175 (General Order), NPDES No. CAG995001; Contra Loma Reservoir, Contra Costa Water District, Contra Costa County" [letter dated June 25, 2009].

New interior Air Valve installed during Phase 1 of the CPL improvement Project on the Tesoro Refinery property.

Typical Air Valve housing, located at the SCPL Site 8.

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Figure 11 - New Air Valve Installation and Typical Existing Air Valve Housing

Source: CCWD

1.2.6 Work Details

Several temporary staging areas or turnarounds will be established along the pipeline or access roads (see Site Maps 1-10 in **Appendix B)**. This will facilitate construction work and materials stockpiling. Temporary construction easements up to approximately 50 feet wide along the SCPL alignment would provide for localized staging of equipment and materials.

Construction activities would generally occur Monday through Friday from 8:00 a.m. to 5:00 p.m. Work on installation of new Butterfly Valves would likely be performed 24 hours a day for several days (until completion). It is anticipated that access road construction would require 14 workers while valve and pipe work would require 16 construction workers.

1.2.7 Construction Scheduling – Operational and Environmental Considerations

Construction of access roads would likely occur during summer or fall to avoid wet weather. Valve replacement and installation would require shutdown of the SCPL, so this work would need to occur when the Loop Canal can handle expected water demands. The Loop Canal is typically out of service for cleaning from November to March. There are no operational based restrictions for settlement monitor or cathodic protection work, although this would likely coincide with valve repair work.

In addition to operational constraints, there are other site-specific scheduling restrictions based on environmental factors considered in **Section 3**. Access road construction is subject to the most significant constraints as it involves the greatest disturbance of wetlands and habitat. It is expected that the construction of access roads will facilitate subsequent or concurrent valve and settlement monitor work within the access roads footprint by reducing the potential for environmentally based schedule restrictions. **Table 4** presents environmentally based calendar constraints and the possible schedule based on these constraints for road construction at each site. Sites 3, 4, 7, and 10 are the most constrained since road construction would take place in wetlands and also would have to allow for the presence of nesting birds. At these sites, the possible work window is very narrow and would need to occur from September 1 to October 15. At Sites 4 and 5, road construction would not be subject to constraints from nesting birds. Work there could proceed from April 15 through October 15. Site 6, located on Tesoro property on the WMU4 site, has the least habitat and wetland constraints however this site does have potential for hazardous waste.

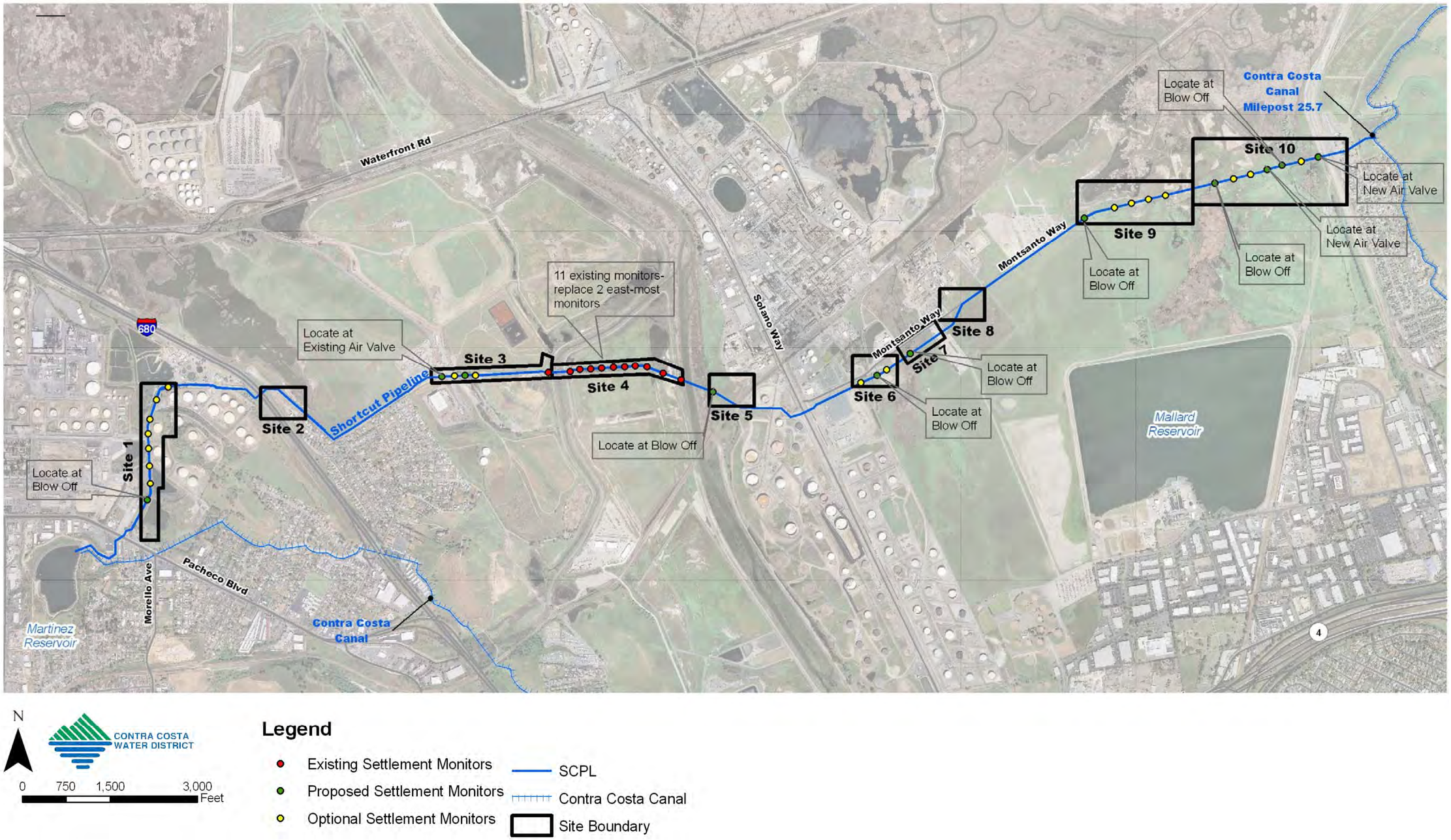

Figure 12 - Proposed Locations of New Settlement Monitors

Source: CCWD

Example of an existing settlement monitor, located at SCPL Site 4.

Figure 13 - Typical Settlement Monitor (Site 4) **Source: CCWD**

Table 4 Road Construction Calendar Constraints and Possible Schedule				
Site No.	**Nesting Bird (Avoidance Window Feb 1-Aug 31)**	**Wetlands Work (Allowed Apr 15-Oct 15)**	**Pickleweed Present (No Calendar Constraint)**	**Schedule**
3	X	X		Sep 1-Oct 15
7	X	X		Sep 1-Oct 15
10	X	X	X	Sep 1-Oct 15
4		X	X	Apr 15-Oct 15
5		X	X	Apr 15-Oct 15
6				Anytime

Once access roads have been constructed, valve and settlement monitor work can begin. It is possible that valve work can commence at the same time as the road work. There are 3 sites where there is currently access and no road construction is planned. Valve work is subject to the operational constraint discussed above. Valve and settlement monitor work is not subject to the wet weather restrictions as the excavation is minor. In addition, many of the valves and settlement monitors will fall within the footprint of the new access roads, so there will be no wetland impact. The driving constraint for any valve work is the need to take the SCPL out of service. There are a few sites, where nesting birds could hinder valve or settlement monitor work.

Figure 14 below illustrates the environmental constraints for sites construction and the possible schedules based on these combined constraints. The majority of work along the pipeline must be completed between September-December. Some sites have added flexibility, and work at these locations can also be completed in April and May.

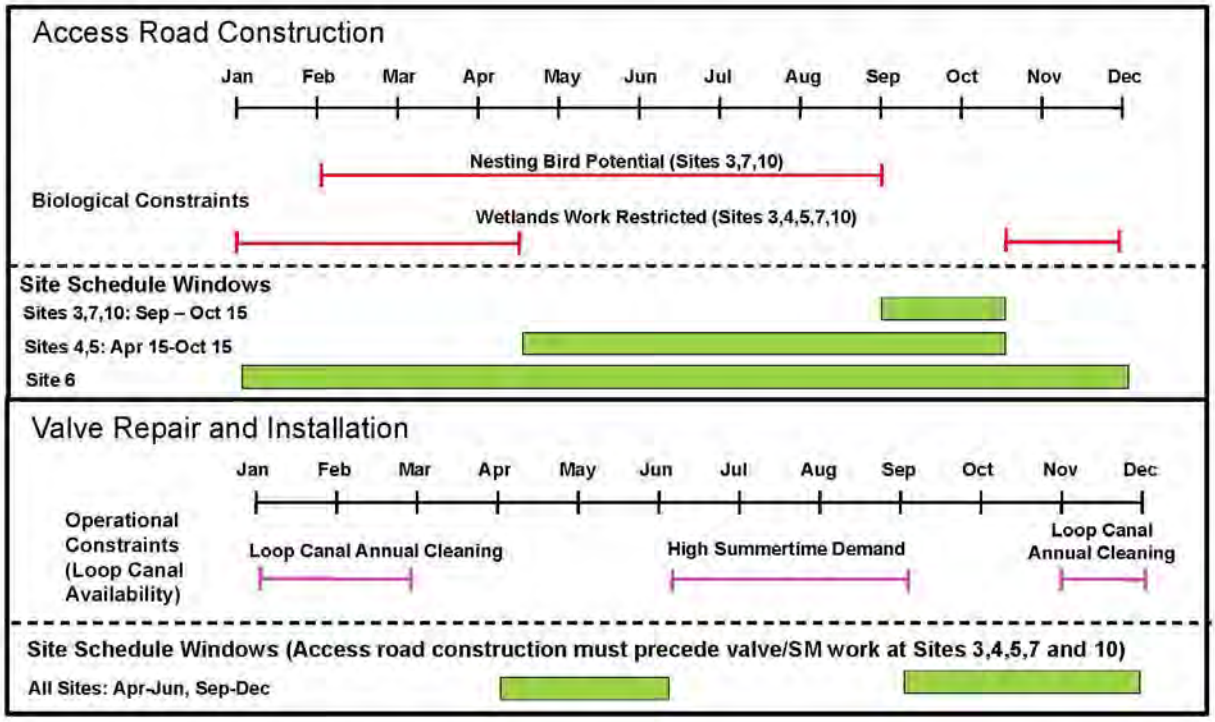

Figure 14 – Calendar-based Environmental Constraints **Source: CCWD**

1.2.8 Phase 3 Pipe Inspection and Repair

Phase 3 of the proposed Project would entail inspection and repair of the SCPL. Following completion of Phase 2 of the proposed project, the SCPL interior would be visually inspected for damage, particularly the area where the prior break occurred as a result of the 1989 Loma Prieta earthquake, located on Site 4. It is possible that some sections of pipe located between Walnut Creek on the east and Site 3 are cracked and would need to be repaired or replaced. This would entail excavation of an area—centered on the pipeline—of about 10 feet wide and 10 feet deep, with length varying depending on the length of damaged pipe section. The cracked pipe sections would be wrapped and sealed and the pipe will be stabilized with crushed stone, then backfilled with compacted soil.

Under a worst-case scenario, an approximately 2,100-foot-long SCPL section between Walnut Creek and Pacheco Creek would be replaced. A 10-foot-wide trench would be excavated by backhoe and excavator following dewatering of the pipeline. Replacement pipe would be laid on a crushed stone foundation, then the trench would be backfilled with compacted soil.

1.2.9 Future Operations and Maintenance

Reclamation worked with the USFWS on operations maintenance activities for a wide range of facilities in the Central Valley Project including the SCPL.[6] Carrying out repairs and maintenance activities of the SCPL requires undertaking numerous activities. The District has identified necessary SCPL operations and maintenance (O&M) activities (designated by activity number) from Table 1 of the USFWS BO. They are listed and described below.

Recommended O&M Activities Associated with the Shortcut Pipeline[7]

Activity 2: Blading and Disking of Right-of-Way. A grader or tractor with mounted blade or disc is used to scrape or shallowly till the soil to kill, prevent, or retard growth or spread of weeds, to reduce cover for pests, and to limit vegetation fuel load while providing fire breaks. This activity is conducted once to several times per year and may be conducted at any time of year, but primarily occurs during the dry season (March through November).

Activity 3: Blading of O&M Roads. A grader or tractor with mounted blade is used to scrape unpaved roadways and road shoulders to remove weedy vegetation, ruts, and to level and maintain the surface for access to the project. This activity is conducted once annually during the dry season, primarily from May through November.

Activity 5: Canal/Tunnel/Conduit Liner Repair. Cracked or broken pipe liner panels or the aprons or outlets at the Canal are patched with concrete, grout compound, shotcrete, or other similar material that is pumped, blown, or fed from a mixture by gravity. Damaged liner that cannot be repaired is overlaid with shotcrete or removed with heavy equipment and a new panel is fashioned in place.

Activity 8: Contact Herbicide Applications. Contact herbicides are applied to control vegetation on canal banks, on rights–of–way, and around water intakes and other structures. Contact herbicides are sprayed from pressurized tanks via

[6] Biological Opinion of the U.S. Fish and Wildlife Service, 2005. Formal Endangered Species Consultation on the Operations and Maintenance Program Occurring on Bureau of Reclamation lands within the South-Central California Area Office, U.S. Fish & Wildlife Service, February 17, 2005.

[7] The District will perform pre-maintenance biological surveys as necessary.

vehicle–mounted booms, backpack sprayers or other application rig, bean gun, wand, or by manually wicking herbicides directly onto vegetation.

Activity 11: Drain Ditch and Channel Maintenance. Debris, trash, soil, sediment, and vegetation are cleaned from open ditches, canals, basins, or pipe in order to ensure the conveyance of water through facilities were a way from facilities. Material may be removed by hand, shovel, backhoe, gradall, excavator, or tractor. Cleared soil and vegetation may be piled on adjacent land or, when extensive, transported in trucks to spoil site. This activity is conducted annually, primarily in spring and fall, but concentrated during the end of the dry season (August through October).

Activity 13: Hand Control of Vegetation. Small amounts of nuisance vegetation and/or weeds are removed at facilities or around structures where use of equipment or herbicides is impractical. Removal is done by hand pulling or with aid of stringed weed cutters, spades, hoes, shovels, adzes, saws, or other hand implements. This activity is conducted year–round.

Activity 14: Insecticidal Sprays. Insecticides are applied year–round, as needed, but primarily from spring through fall, to control bees, wasps, spiders, ants, cockroaches, fleas, termites, mosquitoes, and other arthropods. They are applied directly from canisters, applicators, or by hired structural pest control specialists. They are applied at structures along conveyance facilities and appurtenant structures.

Activity 15: Mudjacking/Injecting Grout. Mudjacking and/or injecting grout is used to repair leaks, cracks, holes, or voids in the canal or pipeline in order to prevent a blowout failure that could result in flooding of surrounding land. Repairs are conducted when defects are discovered, with work preferentially conducted during dewatering. Holes are bored behind the liner with an auger, then grout or fill (liquefied clay) is gravity fed from a mixer through tubes or hole borings into the void until the void is filled.

Activity 16: Pre-emergent Herbicide Applications. Pre–emergent herbicides are used where nearly year–round invasive weeds threaten the facility integrity or increase fire hazards through the growth of fuel load. The herbicides are applied directly to soil before seeds germinate, usually once annually in fall or early winter. Applications are made from pressurized spray tanks with a vehicle–mounted boom sprayer, backpack sprayer, or, for granular formulations, with spreaders. Pre-emergent herbicides are applied around water intakes, on canal banks, on rights–of–way, and around structures.

Activity 18: Right-of-Way Dust Abatement. Dust abatement is conducted to minimize the fugitive dust where the unpaved, non-operational roadway or outer canal bank is graded and where construction is occurring or spoils soil is being hauled during work operations at facilities. Typically, a water truck traverses the roadway or work area and sprays water directly onto the soil surface during single or multiple passes. Flooding may also be used to limit dust.

Activity 19: Right-of-Way Mowing. Mowing is conducted primarily in spring to control weeds and reduce or eliminate the need for herbicide applications along the canal and adjacent rights–of–way. Mowing is conducted with a rotary, sickle bar, or other mower blade attached to a tractor.

Activity 20: Rip-Rap. Rip–rap is used to prevent erosion of shorelines or embankments, and to strengthen the canal. Rip–rap is comprised of large rocks and boulders of varying sizes that are placed at dams, spillways, and canal or levee banks, especially near bridges and canal undercrossings or water control structures. Rock is delivered to the site by truck and trailer; dumped rock is piled with the aid of backhoes and excavators. The work is conducted when needed to protect banks, but is preferentially performed during the dry season.

Activity 23: Bargate/Fence Installations. Gate and fence installations and repairs are made to limit access to facilities, to provide security and safety, and protect resources where encroachment is a problem. Chain-link fence is installed to restrict public access and prevent dumping or vandalism. Bargates are installed where canal rights–of–way intersect public roadways, such as corners of bridges, on secondary or primary roads, or on parallel fences at or near structures. Holes for fence supports are dug by hand implements, power auger, or backhoes. Holes for bargates are dug with power augers or a backhoe. Pipe rods to which chain–link fencing is attached are set in the ground with concrete. Gates are cleaned and painted.

Activity 29: Drainage Improvements. Heavy equipment, including dozers, tractors, backhoes, longsticks, graders, etc., is used to excavate drainage trenches and install drain pipe or to fill low spots to improve drainage. Trenches and drains are cleared of vegetation and silt with heavy equipment or by hand. Excavated material is piled on levees rights–of–way, or is transported by truck to an off-site location. Drainage improvements are made as needed, though mostly annually, and preferentially during dry conditions—usually in the fall, before rains begin.

Activity 30: Electrical Repairs by Utility Companies. Electrical repairs are made year-round on an as–needed basis by utility line crews operating from service vehicles. Repairs (or replacements) are made at all utility–serviced facilities, including power poles, transformers, and underground utility lines.

Activity 31: Embankment Maintenance. Fill embankments along the sides of the canal are used to reduce runoff and erosion of soil into the canal and/or divert water toward underdrains or overchutes. The embankments are maintained with backhoes, graders, excavators, or hand implements employed to fill gullies and burrows, compact soil, and grade slopes. Trucks are used to haul fill. Embankment maintenance is conducted as needed, but primarily during the dry season.

Activity 32: Facilities Inspection. All project facilities are inspected once annually. Mechanical equipment (gates, valves, etc.) and electrical equipment (communications, monitoring, computer systems, etc.) are visually inspected and operated to test functioning.

Activity 33: Graffiti Removal from Concrete Structures. Graffiti is painted over by hand with a brush or roller, or is removed by sandblasting. Waste paint from sandblasting is collected and disposed of at an appropriate waste disposal site.

Activity 35: Valve Rehabilitation. Valve function is checked annually, and when they do not operate, they are removed and repaired or replaced. Valve rehabilitation work is contained within the existing SCPL ROW.

Activity 41: Right-of-Way Trash Removal. Tires, plastics, lumber, bedding, scrap metal, and other trash and garbage is removed by hand from rights–of–ways and hauled by truck to an appropriate waste disposal site. Larger items such as vehicles and appliances are removed with the aid of service trucks with hoists or winches. Where needed, trained waste handlers are used to collect and dispose of hazardous wastes.

Activity 42: SCADA System Repair and Upgrade. Supervisory Control and Data Acquisition (SCADA) electrical, computer, or communications equipment— primarily modular components or panels— located at control structures is repaired or replaced as needed.

Activity 48: Utility Trenching. Utility trench is excavated with a trencher, backhoe, or excavator to lay underground utilities to facilities and upgrade the systems in place. Most utility infrastructure has been provided, and the need for trenching is infrequent.

Activity 54: Minor Road Construction/Rehabilitation. Minor road rehabilitation or construction is done to provide new access to facilities or to recondition existing roads. Rehabilitation can involve ripping and removal of existing asphalt, re-grading of roadbed, and compaction of the new bed and underlying soil. Sand is spread by truck, along with crushed rock, and new asphalt that is compressed. Road construction is done irregularly or annually on a limited scale.

Activity 57: Structure Construction. Structures (blockhouses, stilling wells, etc.) are constructed when new operational facilities are added. Sites are graded and forms set for pouring concrete pads framing may use concrete block, metal, or would, with metal siding. Trenching may be done to provide underground utilities to the site.

Activity 58: Utility and Facilities Repair. Utility companies may send a service vehicle to repair or electrical connections or replace transformers. Repairs may occur anywhere along facilities, but are primarily conducted inside structures, and are completed irregularly on an as–needed basis.

Activity 59: Pump-In System Set-Up During Flood Years. During flood years, pumped diversion of water into the canal may occasionally be required, as permitted by operational agreements. Pump sites are cleared and leveled with heavy equipment and temporary piping is laid.

Additional ongoing maintenance activities not included above would include regular inspection and maintenance of the cathodic protection system, which protects the external surfaces of the mortar-coated steel cylinder of the SCPL from corrosion. The system consists of seven deep-well ground beds with high silicon, cast iron anodes connected to pole mounted rectifiers, and test stations installed at various locations along the pipeline as shown in **Figure 15**. District operations and maintenance staff would take system power input and output readings on a monthly basis and perform a pipe-to-soil potential survey annually to test the protective potential of the cathodic protection system.

1.3 Required Permits and Approvals

1.3.1 Reclamation Actions

The Proposed Action requires permission from Reclamation to construct the improvements and to maintain the SCPL and its right–of–way. Reclamation must issue the District an MP-620 permit prior to the start of construction of Phase 2, and must confirm that the District is authorized to perform routine maintenance of the refurbished, replaced, and new valves along the SCPL easement. Additionally where new ROWs are required, Reclamation may need to authorize these additional easements in accordance with the amended BO which allows activities outlined in Section 1.2.9. Before Reclamation can authorize the above action, it must prepare an Environmental Assessment under the National Environmental Policy Act (NEPA). It is anticipated that Reclamation will prepare this document as soon as the CEQA document is completed.

1.3.2 Additional Permits

The following permits from other federal, State, regional, and local agencies would also be required:

Federal

U.S. Army Corps of Engineers (COE): Construction of new access roads would require a Section 404 fill permit from the San Francisco Bay COE in compliance with the Clean Water Act (1972). Due to the amount of wetlands or Waters of the U.S. that would be filled (1.23 acres, plus up to 6.49 acres on a temporary basis), the Proposed Action would require an Individual 404 Permit from the COE. The District will formally apply for the 404 permit once the CEQA document has been issued.

U.S. Fish and Wildlife Service (USFWS): As part of its permitting process, Reclamation would initiate a Section 7 consultation with USFWS. The USFWS would make a determination as to whether the Proposed Action could result in biological jeopardy to a plant or animal species listed as Threatened or Endangered under the Endangered Species Act (1973), and if so, identify mitigation measures to protect the affected species and/or offset potential impacts to such species. Listed species are taxa for which proposed and final rules have been published in the Federal Register

San Francisco Bay Area Regional Water Quality Control Board (SFRWQCB): The Project would also require filing of a Notice of Intent (NOI) to the San Francisco Bay Area Regional Water Quality Control Board and preparation of a Stormwater Pollution Prevention Plan (SWPPP). The SWPPP addresses control of storm water pollution during construction through implementation of Best Management Practices (BMPs). In addition, the Project would require Section 401 Water Quality Certification from the RWQCB, pursuant to the federal Clean Water Act, as a prerequisite to a permit from the COE. A National Pollutant Discharge Elimination System (NPDES) permit will not be required for discharge of groundwater from dewatered excavations, because the work is considered maintenance of an existing utility line to restore and maintain its original purpose. Maintenance also includes work to restore existing facilities including access roads. The only work that may be subject to a

General Construction NPDES permit would be the construction of new access roads where none previously existed.

State

California Department of Fish and Game (CDFG): CDFG may require a Streambed Alteration Agreement for any activity that could affect the bank or bed of any stream that has value to fish and wildlife. CDFG authority is typically extended to any "blue line" stream shown on a U.S. Geological Society (USGS) topographic map, as well as unmapped channels with a definable bank and bed.

Local

East Bay Municipal Utility District: (EBMUD): The Project will require an encroachment permit from EBMUD for access to Site 10 that borders the Mokelumne Aqueduct.

Contra Costa County Flood Control and Water Conservation District (CCCFC&WCD): Permission from the CCCFC&WCD is necessary to use levee roads along Pacheco Creek and Walnut Creek for construction to access Sites 3 and 4. A 10-year license agreement was completed in December 2010 tha allows maintenance access to Sites 3 and 4. CCCFC&WCD must also agree to a modification of its levee adjacent to the Blow-off Valve at Site 4. This will allow maintenance crews site access to this valve and a truck turnaround.

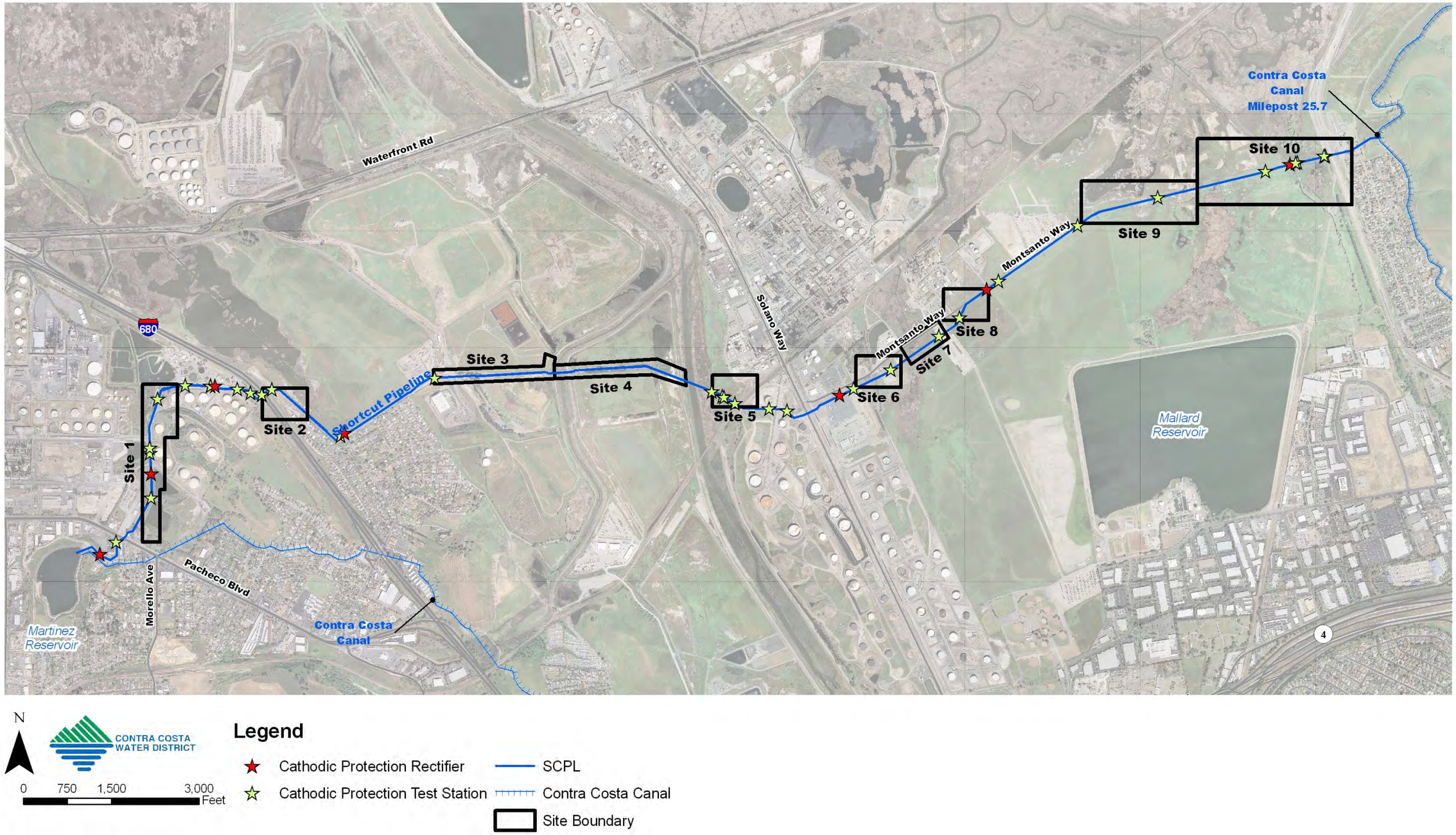

Figure 15 - Cathodic Protection Locations

Source: CCWD

1.4 Potential Environmental Issues

This IS/MND analyzes the affected environment of the Proposed Action in order to determine the potential and cumulative impacts to the following resources. Note that EA refers to a NEPA required section and IS refers to a CEQA required section.

- Aesthetics (IS)
- Agricultural Resources (IS)
- Air Quality (EA/IS)
- Biological Resources (EA/IS)
- Cultural Resources (EA/IS)
- Environmental Justice (EA)
- Geology and Soils (IS)
- Global Climate Change (EA/IS)
- Hazards and Hazardous Materials (EA/IS)
- Hydrology & Water Quality (IS)
- Indian Trust Assets (EA)
- Land Use and Planning (EA/IS)
- Mineral Resources (IS)
- Noise (IS)
- Population/Housing (IS)
- Public Services (IS)
- Recreation (IS)
- Socioeconomic Resources (IS)
- Transportation/Traffic (IS)
- Utilities/Service Systems (IS)

Section 3 includes the analysis of potential environmental issues for both CEQA (IS/MND) and NEPA (EA/FONSI).[8] Reclamation will finalize the EA/FONSI once the IS/MND is completed. **Section 3** includes the analysis portion of the potentially affected environment and the environmental consequences involved with the Proposed Action and the No Action Alternative. Mitigation measures are also indicated in this section where appropriate.

[8] EA/FONSI – Environmental Assessment/Finding of No Significant Impact.

Section 2 Alternatives and Proposed Action

This environmental report considers two possible actions: the No Action Alternative and the Proposed Action. The No Action Alternative reflects future conditions without the Proposed Action and serves as a basis of comparison for determining potential effects to the environment. Other alternatives were considered but rejected from further evaluation due to infeasibility and/or greater environmental impacts. One option entailed relocating portions of the SCPL to avoid impacts to sensitive habitat areas. However, the costs of new land acquisition would be excessive and substantial impacts to sensitive habitat would still occur. Another alternative to facilitate full maintenance access considered construction of a new permanent access road along the entire length of the pipeline. This alternative was rejected because it would have greater impacts than the Proposed Action as well as greater costs.

2.1 No Action Alternative

Under the No Action Alternative, Reclamation would not approve the construction of Phase 2 of the Project. No new access roads would be constructed, no valves would be replaced or added, and no new settlement monitors would be constructed. Operation and maintenance of the SCPL would continue under the currently constrained conditions, with difficult to infeasible access to certain portions of the pipeline. Over time, aging and deteriorated valves could become inoperable, further impeding the feasibility of repairing and maintaining the pipeline. The reliability of the water supply CCWD provides to the City of Martinez would be compromised and could be completely cut-off in the event of a rupture of the pipeline. Reliability of supplies to users along the entire Contra Costa Canal would be compromised because the use of the Martinez Reservoir for balancing daily flows would be adversely affected. Under this scenario, the No Action Alternative could result in a series of adverse secondary effects which are evaluated in this environmental document.

2.2 Proposed Action

The Proposed Action is defined as Phase 2 and Phase 3 of the project. The Proposed Action will construct new access roads in 5 locations along the SCPL alignment to provide access—for maintenance purposes—to valves that are currently inaccessible or accessible only during dry summer months. Once constructed, the roads would allow CCWD to access valves without adversely affecting sensitive biological habitat located in and adjacent to some portions of the alignment.

Following completion of or in conjunction with the proposed access roads, the Proposed Action would include inspection of sections of the pipeline that may ultimately require repairs due to cracking and deterioration in some segments. Additionally, aging valves would be replaced or refurbished, and new valves would be installed to improve efficiency and flexibility for pipeline operations and maintenance.

More specifically, the Proposed Action would include:

- Potential new Rights-of-Way from Contra Costa County and Tesoro.
- Temporary access agreements from Contra Costa County, EBMUD, and the Concord Naval Weapons Station.
- Construction of 5 new gravel at-grade access roads at Sites 4,5,6,7 and 10 totaling approximately 5,000 feet in length, including:

- Approximately 1,900 feet along the easternmost segment of the pipeline (Site 10);
- Three segments within the Tesoro Refinery consisting of approximately 480 feet near the Foster Wheeler power plant (Site 5), 130 feet within WMU4 (Site 6), 580 feet east of WMU4 with-in Site 7 and;
- Approximately 2,000 feet in the area where the pipeline failed during the 1989 Loma Prieta earthquake (Site 4); and
- Construction of minor gravel, paved, or concrete road segments to the remaining valves (3 Air Valves and 3 Blow-off Valves). Regraveling of 480 feet of existing road (Site 9).

- Replacement of 5 existing Air Valves.
- Installation of 2 new Air Valves.
- Replacement of 8 Blow–off Valves.
- Replacement of 2 Butterfly Valves.
- Installation of 1 new Butterfly Valve.
- Installation, replacement, repair of up to 40 settlement monitors.
- Inspection and repair of cathodic protection system as needed.
- Ongoing maintenance of the pipeline, valves, cathodic protection equipment, settlement monitoring stations, and other stations along the pipeline alignment.

Additional details and illustrating maps and figures are provided in **Appendix B**.

HYDROLOGIC SUFFICIENCY ANALYSIS
RHEEM CREEK MITIGATION BANK
RICHMOND, CALIFORNIA

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October 2012

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1.0 Approach and Findings

The Rheem Creek Mitigation Site features the preliminary design of seasonal wetlands and riparian corridor enhancements for a former industrial site adjacent to Richmond Parkway in San Pablo, California. The main objective is to create wetlands that sustain saturation and ponding for up to 18 continuous days during the growing season. Optimal design of seasonal freshwater wetlands for the Rheem Creek Mitigation Site necessitates a hydrologic sufficiency analysis. Using available hydrologic and meteorological data, Kamman Hydrology & Engineering, Inc. (KHE) developed individual water budgets for four watershed basins within the site to optimize the size and function of four proposed seasonal freshwater wetlands. Water budgets were prepared for average, median year-type, and dry year-type precipitation years. Key findings of this study include the following:

- Seasonal freshwater wetlands are sustainable by direct rainfall, even with no other contribution of surface water runoff or groundwater inflow. Wetland creation is therefore controlled by suitable soil type (i.e., high clay content) and level ground.

- Wetland hydroperiods could be extended by introducing supplemental water supplies to seasonal wetlands, such as localized runoff, groundwater, and overbank storm flow from Rheem Creek.

The following Sections summarize the approach, methods, and results of KHE's hydrologic sufficiency analysis.

2.0 Topography, Watersheds, and Proposed Wetlands

The Rheem Creek Mitigation Site lies south of Rheem Creek, within the approximately 13.6 acre project site located between Richmond Parkway to the northwest, the Union Pacific railroad line to the west, AT & SF railroad line paralleling Giant Road to the east, a proposed rail line spur trending southwesterly from Giant Road and the AT & SF railroad alignment, and an existing industrial facility to the south. The proposed Parkway Commerce Center lies on the north side of Rheem Creek. The drainage areas of the proposed wetlands were established from electronic Contra Costa County one foot-interval LIDAR contour maps and the assumed drainage pathway from a contributing portion of Richmond Parkway. Figure 1 illustrates a site map including the proposed wetlands and watersheds. It was KHE's understanding that the existing topography of the site is to be regraded before wetland basin grading commences; therefore locations of the proposed wetlands are conceptual in nature and will likely be reconfigured with the final site grading.

3.0 Rheem Creek Watershed and Streamflow

United States Geological Survey (USGS) operated a stream gauge on Rheem Creek from December 1960 through September 1990 (USGS Gauge 11182030, 2012). The gauge, located just west of Giant Road, and immediately upstream from the Rheem Creek Mitigation Site, measured runoff in the upper 1.49 square miles of the Rheem Creek Watershed. Data available included mean daily flow, mean monthly flow, and yearly peak flow records.

A detailed study of the Rheem Creek Watershed by the National Heritage Institute (NHI, 2007) suggests the watershed upstream of the gauge was nearly completely developed or "built out"

prior to gauge records, and no major changes have been made to the channel since the gauge was discontinued in 1990. It is likely, and assumed in this analysis, that stream flow recorded by the gauge is similar to stream flow in the channel today. The watershed contributing flow at the stream gauge site consists of some limited open space in the headwaters, several residential subdivisions, the campus of Contra Costa College, and a region of industrial development.

Figure 2 illustrates a flow duration curve developed from average daily flow, indicating the percentage of time in which specific flows are equaled or exceeded at the USGS flow gauge. Average daily flows of 10 cfs or less occurred 96% of the time within the period of record. Figure 3 illustrates an annual peak discharge exceedance probability curve for the USGS flow gauge. Using the USGS Bulletin 17B PeakFQ program (USGS, 2007) and yearly peak flows, the program predicts flows expected to result from a specific recurrence interval storm event, such as a 100-year storm. A 100-year storm is predicted to be exceeded in severity only once every one hundred years, on average. Table 1 provides flow predictions for specific flood recurrence interval events.

Table 1. Rheem Creek Discharges at Selected Exceedance Probabilities.

Exceedance Probability / Recurrence Interval	Peak Flow (cfs)
50% (2-Year Storm)	291
20% (5-Year Storm)	360
10% (10-Year Storm)	396
4% (25-Year Storm)	435
2% (50-Year Storm)	460
1% (100-Year Storm)	481

4.0 Precipitation and Water Year Types

The National Climatic Data Center (NCDC) of the National Oceanic and Atmospheric Administration (NOAA) records daily precipitation for Richmond, California (NCDC Precipitation Station USC00047414, 2012). Daily and monthly data extends from December 1950 through 2012. The precipitation gauge is approximately 4 miles southwest of the location of the Rheem Creek stream flow gauge and the Rheem Creek Mitigation Site.

Data was organized into Water Year[1] (WY) 1951 through 2011. The long-term average annual rainfall estimate from the data is 23.2 inches, based on a daily rainfall summation of all available data. The value agrees well with the USGS estimate for mean annual rainfall of 21-inches for this location (Rantz, 1971).

Monthly total rainfall values were used to develop a rainfall-runoff year-type probability analysis. Monthly rainfall values for WY 1951 through 2011 were summed by water year and ranked. Any specific water year missing at least one month of data was removed from the analysis. The exceedance probability ranking of the remaining annual rainfall values suggests the long-term yearly average value of 23.2 inches has a 45% probability of occurring any given

[1] The USGS defines a water year as the 12-month period beginning October 1st for any given year, through September 30th of the following year. The water year is designated by the calendar year in which it ends and which includes 9 of the 12 months. Thus, the year ending September 30, 1999 is called the "1999" water year.

year. Thus, the statistical average value does not equal the median value. The median year in the data set, or that with a 50% probability of occurring within any given year, is WY 1984; data for this year was used for the median year-type analysis. WY 1984 generated 20.9 inches of total annual rainfall. A water year-type with an 84% probability of occurring within any given year was selected as the representative dry year-type. The water year exhibiting 84% probability of occurring within any given year is WY 1961, which generated 14.4 inches of total annual rainfall. Average, median year-type, and dry year-type monthly rainfall totals are presented in Tables 2, 3 and 4, respectively.

5.0 Runoff

Rainfall data was correlated to with stream flow data (rainfall-stream flow method) for WY 1961 through 1990 to estimate monthly runoff. The monthly average rainfall volume for WY 1961 through 1990 was converted to acre-feet from inches by multiplying by the watershed area contributing to the USGS stream gauge (1.49 square miles). The monthly average stream flow volume was converted to acre-feet from cfs/day. The same comparison was made for the median water year-type (WY 1984) and the dry year-type (WY 1961), as well as alternate median and dry year-types (WY 1971 and WY 1975 median, WY 1981 dry). A ratio of runoff (stream flow) volume to rainfall volume on a monthly basis was computed. The rainfall-stream flow method produced a summation of average monthly runoff to rainfall ratio for WY 1961 – 1990 of almost 60%.

Runoff was also calculated using the Natural Resource Conservation Service (NRCS) Runoff Curve Number (CN) Method (NRCS, 1986). The CN method approximates volume of direct surface runoff as a function of daily (24 hour) rainfall (P), the potential maximum retention after runoff begins (S), the initial abstraction (I_a), and the curve number (CN). Estimated as 20 percent of the value for S, I_a accounts for the total water losses before runoff begins and includes depression storage, interception, evaporation, and infiltration. S is directly related to CN, a function of hydrologic soil group (HSG), cover type, treatment, hydrologic condition, and antecedent runoff condition.

Soil data (NRCS, 1977) in the upper and middle watershed consist of moderately well-drained loams and clay loams, and cut and fill soils, a result of extensive human manipulation, with slow to very slow permeability. The extent of development in the watershed suggests choosing HSG D with low infiltration and high runoff potential.

Based on a chosen CN value of 95, the parameters to the runoff equations were estimated as:

$S = (1000 / CN) – 10 = 0.53$ inches
$I_a = 0.2 * S = 0.11$ inches

These data indicate that within a 24-hour period, the initial 0.11 inches of rainfall goes towards depression storage, interception, evaporation, and infiltration. Below this initial rainfall total, no runoff occurs. Rainfall in excess of 0.11 inches generates surface water runoff (Q) by the equation:

$$Q = ((P – 0.2S)^2) / (P +0.8S)$$

Using the NRCS method, the yearly summation of average monthly runoff values over the entire recorded precipitation period (WY 1951 – 2011) was approximately 10.6 inches, or approximately 46% of the summation of average monthly rainfall value of 23.2 inches. The yearly summation of average monthly runoff values over the stream gauge recording period (WY 1961 – 1990) was approximately 10.1 inches, or approximately 45% of the long term yearly average rainfall value of 22.4 inches for that time period.

Monthly data over October through March (wet months) for both methods indicated a range of ratios from 40% to 70% for the rainfall-stream flow method and 35% to 50% for the NRCS method. The same comparisons for the median year-type and dry year-type were less conclusive due to the variability of a single specific month's rainfall volume.

Based on the ratios described above, an estimate of volume of monthly runoff as 50% of a volume of monthly rainfall was used in the analyses. Analyses were also completed assuming the wetlands received water from rainfall only with no contributing runoff. Average, median year-type and dry year-type monthly runoff totals are presented in Tables 2, 3 and 4, respectively.

6.0 Evapotranspiration

Monthly pan evaporation was not available for a site near the Rheem Creek Mitigation Site. Instead, evapotranspiration data was obtained from a map of reference evapotranspiration produced by the California Irrigation Management Information System (CIMIS, 1999). The site is located in Zone 2, the "Coastal Mixed Fog Area." A multiplier of 1.1 was applied to estimate evapotranspiration from a wetland as opposed to an open water surface. Average, median year-type, and dry year-type monthly evapotranspiration values were assumed the same, and are presented in Tables 2, 3 and 4.

Table 2. Average Monthly Input Values, December 1950 – May 2012.

	Rainfall (inches)	Runoff (inches)	Evapotranspiration (inches)
October	1.30	0.65	3.07
November	2.90	1.45	1.98
December	4.34	2.17	1.36
January	4.83	2.42	1.36
February	3.90	1.95	1.85
March	3.24	1.62	3.41
April	1.66	0.83	4.29
May	0.51	0.25	5.12
June	0.20	0.10	5.61
July	0.04	0.02	5.46
August	0.07	0.03	5.12
September	0.23	0.12	4.29
Annual	**23.20**	**11.60**	**42.91**

Table 3. Median Year-Type (50th Percentile of Being Equaled or Exceeded) Input Values, WY 1984.

	Rainfall (inches)	Runoff (inches)	Evapotranspiration (inches)
October	0.45	0.23	3.07
November	7.16	3.58	1.98
December	7.49	3.75	1.36
January	0.31	0.15	1.36
February	1.66	0.83	1.85
March	2.29	1.14	3.41
April	1.19	0.59	4.29
May	0.01	0.01	5.12
June	0.12	0.06	5.61
July	0.00	0.00	5.46
August	0.16	0.08	5.12
September	0.05	0.03	4.29
Annual	**20.89**	**10.44**	**42.91**

Table 4. Dry Year-Type (84th Percentile of Being Equaled or Exceeded) Input Values, WY 1961.

	Rainfall (inches)	Runoff (inches)	Evapotranspiration (inches)
October	0.26	0.13	3.07
November	3.43	1.71	1.98
December	2.61	1.31	1.36
January	1.91	0.95	1.36
February	1.52	0.76	1.85
March	2.79	1.40	3.41
April	1.14	0.57	4.29
May	0.35	0.18	5.12
June	0.00	0.00	5.61
July	0.00	0.00	5.46
August	0.05	0.03	5.12
September	0.29	0.15	4.29
Annual	**14.35**	**7.17**	**42.91**

7.0 Modeling Approach and Wetland Optimization

In order to optimize the four wetlands to provide the desired seasonal wetland wet-dry cycle, the following success criteria were applied to the analysis. First, each wetland must achieve two consecutive months at 75% volume, or 0.75-foot depth during median water year-type with 50% chance of rainfall being equaled or exceeded in any given year. A second objective is to verify that at least two consecutive months at 75% volume, or 0.75-foot depth during median water

year-type, are met with no runoff entering the wetland from the surrounding watershed. Maximum available water depth of each wetland area is 1.0-foot, with vertical side slopes.

8.0 Water Budget Analyses

The water budget analysis consisted of processing monthly inflow, outflow, and storage volume changes for each wetland. A typical wetland water budget consists of tracking the volume of inflow into the wetland and overflow from the wetland into Rheem Creek or an adjacent watershed.

A typical water budget for a wetland system accounts for the monthly inflows, outflows and changes in wetland storage as described below. Water budgets were computed for each wetland under average, median year-type, and dry year-type conditions.

- Monthly direct rainfall inflow is converted from inches to volume (acre-feet) by multiplying monthly rainfall by the wetland area[2].

- Monthly surface water runoff inflow is converted from inches to volume (acre-feet) by multiplying monthly runoff from the contributing watershed area, excluding wetland area (net drainage area). The analyses were also performed with no contributing surface water runoff inflow to the wetlands.

- Monthly evapotranspiration outflow is converted from inches to volume (acre-feet) by multiplying the first month (October) by the "empty" wetland surface area. Each successive month's evapotranspiration volume is multiplied by the previous end of month wetland surface area.

- Accounting for the rainfall, runoff, and evapotranspiration volumes for each month produces the monthly wetland inflow balance. The monthly inflow balance is positive if the sum of rainfall and runoff exceed evapotranspiration losses. Or, the monthly inflow balance may be zero when evapotranspiration volume is greater than contributing rainfall and runoff volumes. Monthly inflow is added cumulatively, month by month, with any negative monthly values converted to zero to account for dry months.

- End-of-month wetland storage is calculated during filling and draining sequences based on the wetland stage-area-volume relationships derived for each wetland. Outflow or spillage from the wetland is quantified should inflows exceed pond capacity and converting any negative monthly values to zero, accounting for the months where the wetland dries out. End of month stage (wetland water depth) and wetland surface areas are calculated from wetland volume using stage-area-volume relationships. The end of month wetland surface area is used in the water budget to calculate the amount of evaporation occurring in the following month.

- Should spillage from the wetland occur once wetland capacity is exceeded, the monthly spillage volume could be accounted for as an inflow volume to adjacent wetland or

[2] All direct rainfall enters the wetland, and the wetland is assumed to have vertical side slopes.

Rheem Creek, or the wetland could be graded deeper than 1-foot depth. In this analysis, spillage is "lost" to Rheem Creek.

9.0 Results

In general, there is sufficient water supply from rainfall alone to successfully create the four seasonal wetlands totaling 4.82-acres with a water depth of 0.75-foot for at least two consecutive months. Accounting for runoff allows the wetlands to be deeper; all four wetlands achieve at least two consecutive months with 0.75-foot depth. Figures 4A through 4D illustrate each seasonal wetland's water budget results for average, median year-type, and dry year-type rainfall. During dry year-types that include contributing runoff, maximum wetland depths are not achieved, but all wetlands except Wetland 3 would sustain at least 0.5-foot of standing water for four months or more. Wetland 3 would sustain at least 0.25-foot of standing water for four months or more.

10.0 Rheem Creek Overflow Potential

A simple HEC-RAS hydraulic model of Rheem Creek under existing conditions was created using four cross-sections as model geometry. The steady state model used the 2-Year, 5-Year, 10-Year, 25-Year, 50-Year, and 100-Year recurrence interval storm event flows to determine if high flows within the creek could supply water to the four wetlands. Figure 5 is a representative cross-section with six simulated flood water levels located approximately 225 feet downstream of Giant Road and directly adjacent to the site.

Despite the existence of a levee on the south side of Rheem Creek, water may overtop the upstream portion of the levee during a 2-Year storm event carrying 291 cfs. Further downstream, the presence of the levee may trap the overflow water onto the site. Thus, during future site and wetland grading, overbank flow from Rheem Creek can be integrated as potential water supply to selected project wetlands.

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0" 1"

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MARK	DATE	DESCRIPTION
		ISSUE

RHEEM CREEK

CONTRA COSTA COUNTY

PROPOSED PLAN VIEW

PROJ. NO.: 3112 / 7112

DRWN: SGL CHKD: GRK

1

SHEET 1 OF 1

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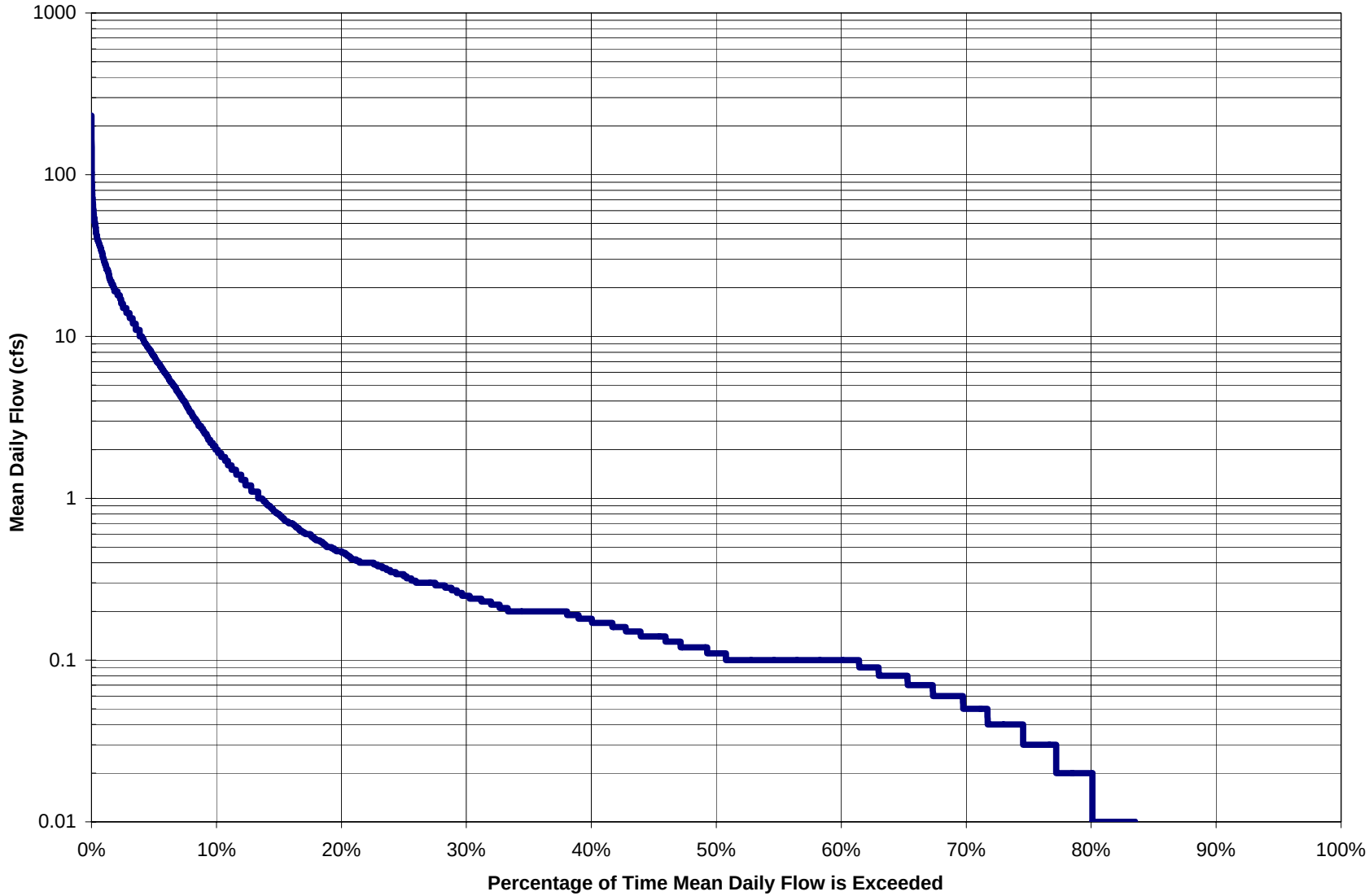

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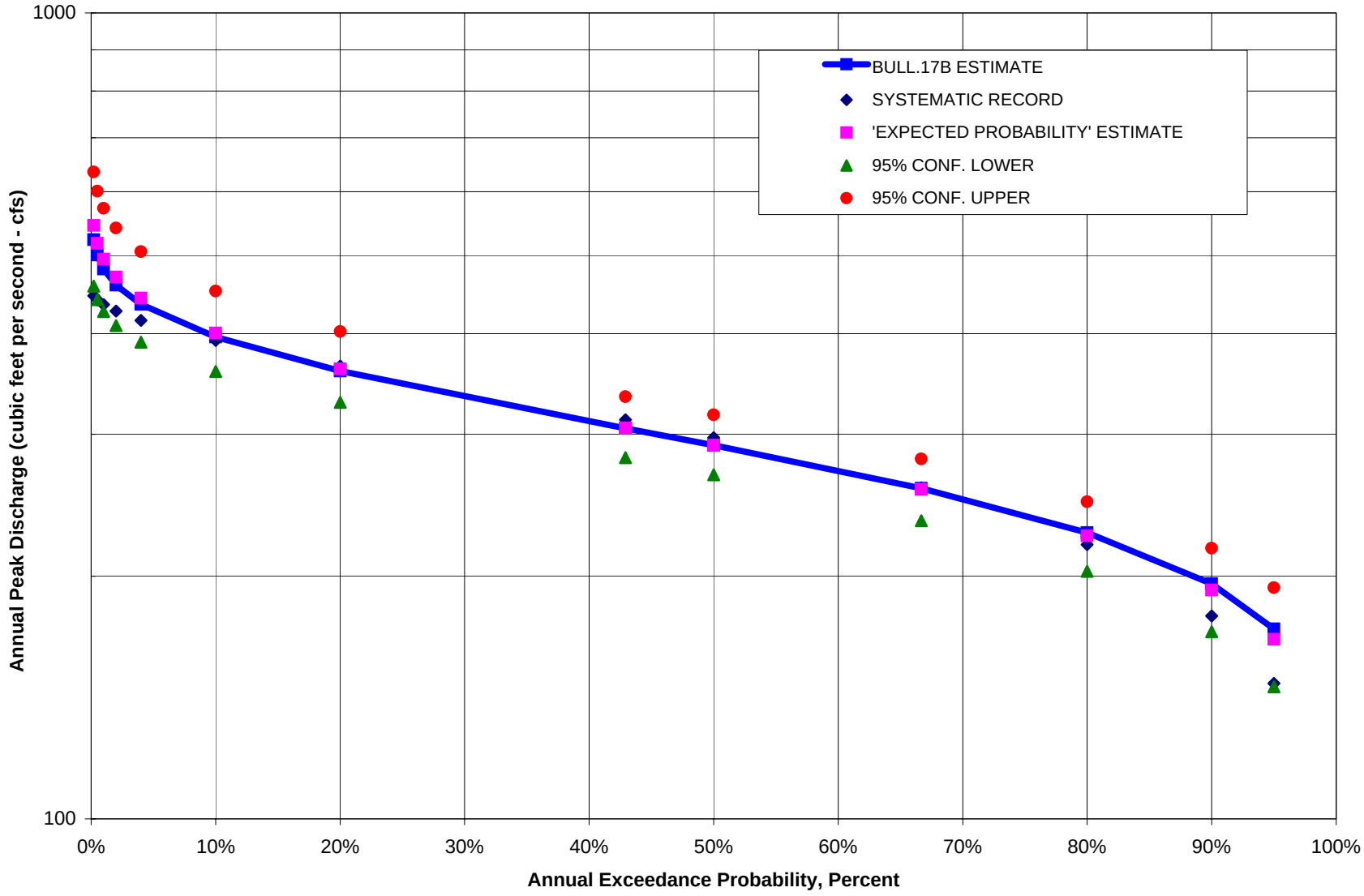
Rheem Creek Mean Daily Flow Duration Curve

12/01/1960 - 09/30/1990

FIGURE 2

Z:\3112_Olberding\7112 Rheem-Cr\Flow\Rheem Cr Mean Daily and Flow Duration



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Rheem Creek Annual Exceedence Probability

1961 - 1990

FIGURE 3

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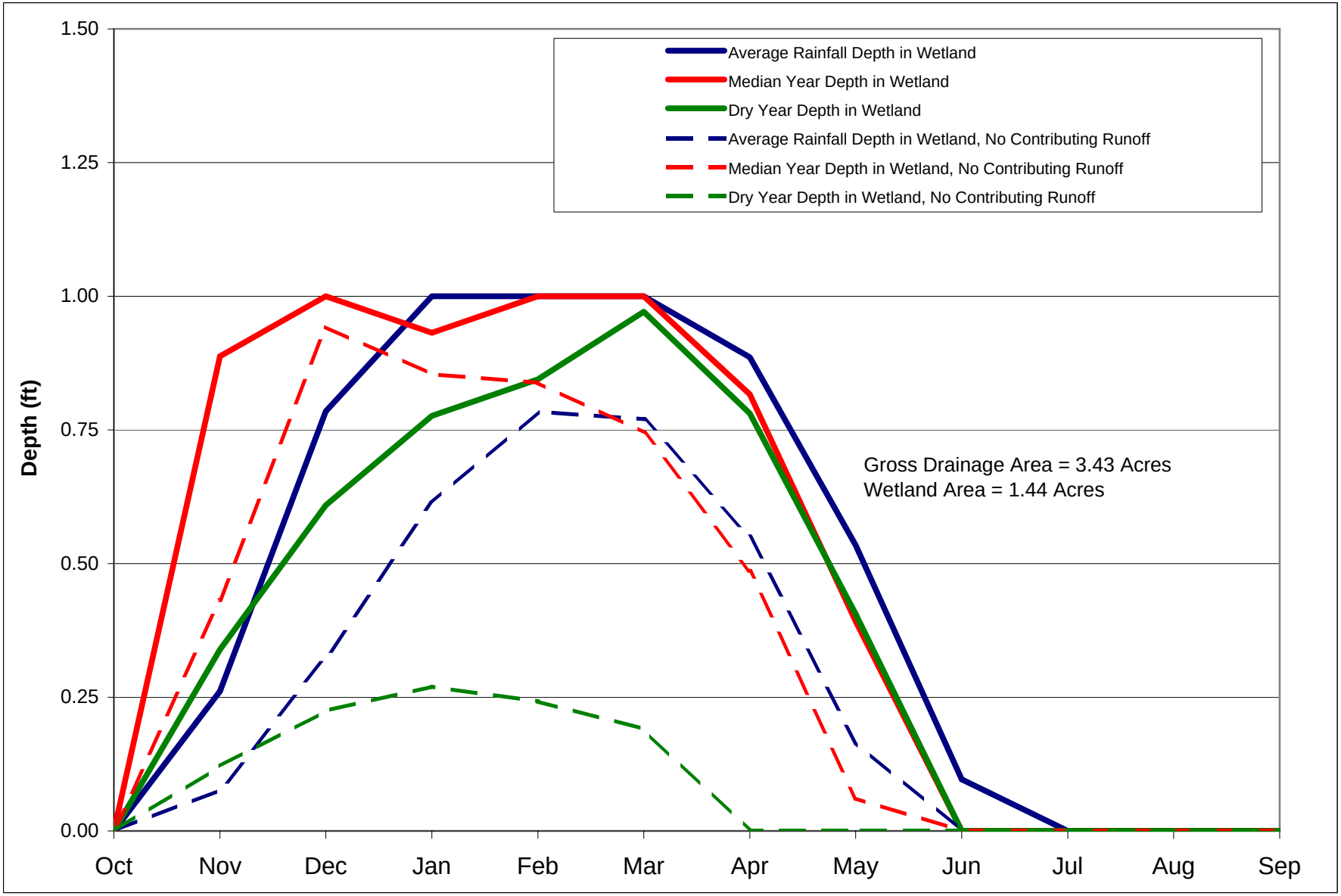

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Water Budget Results

Watershed A + Wetland 1

FIGURE 4A

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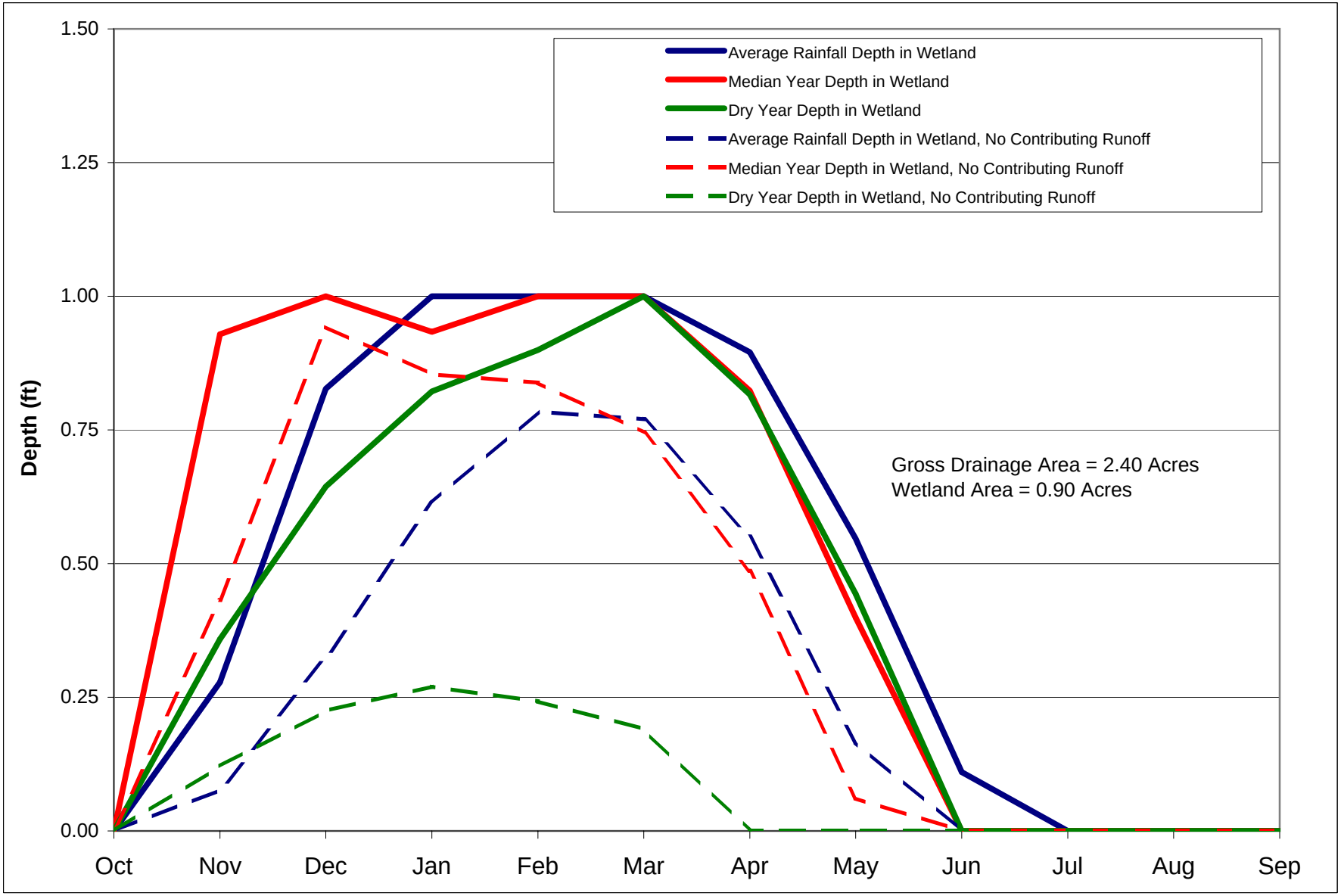

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Water Budget Results

Watershed B + Wetland 2

FIGURE 4B

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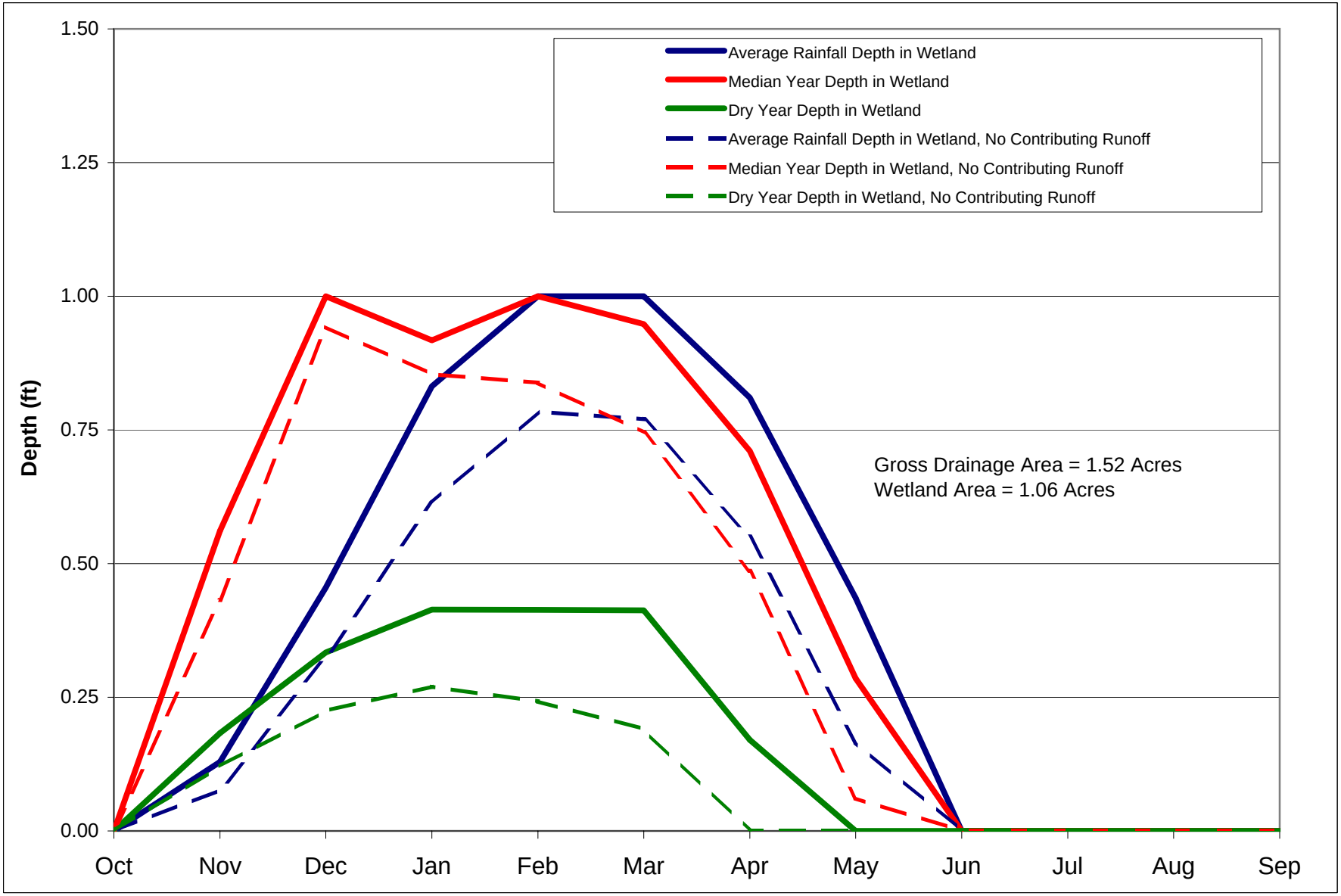

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Water Budget Results

Watershed C + Wetland 3

FIGURE 4C

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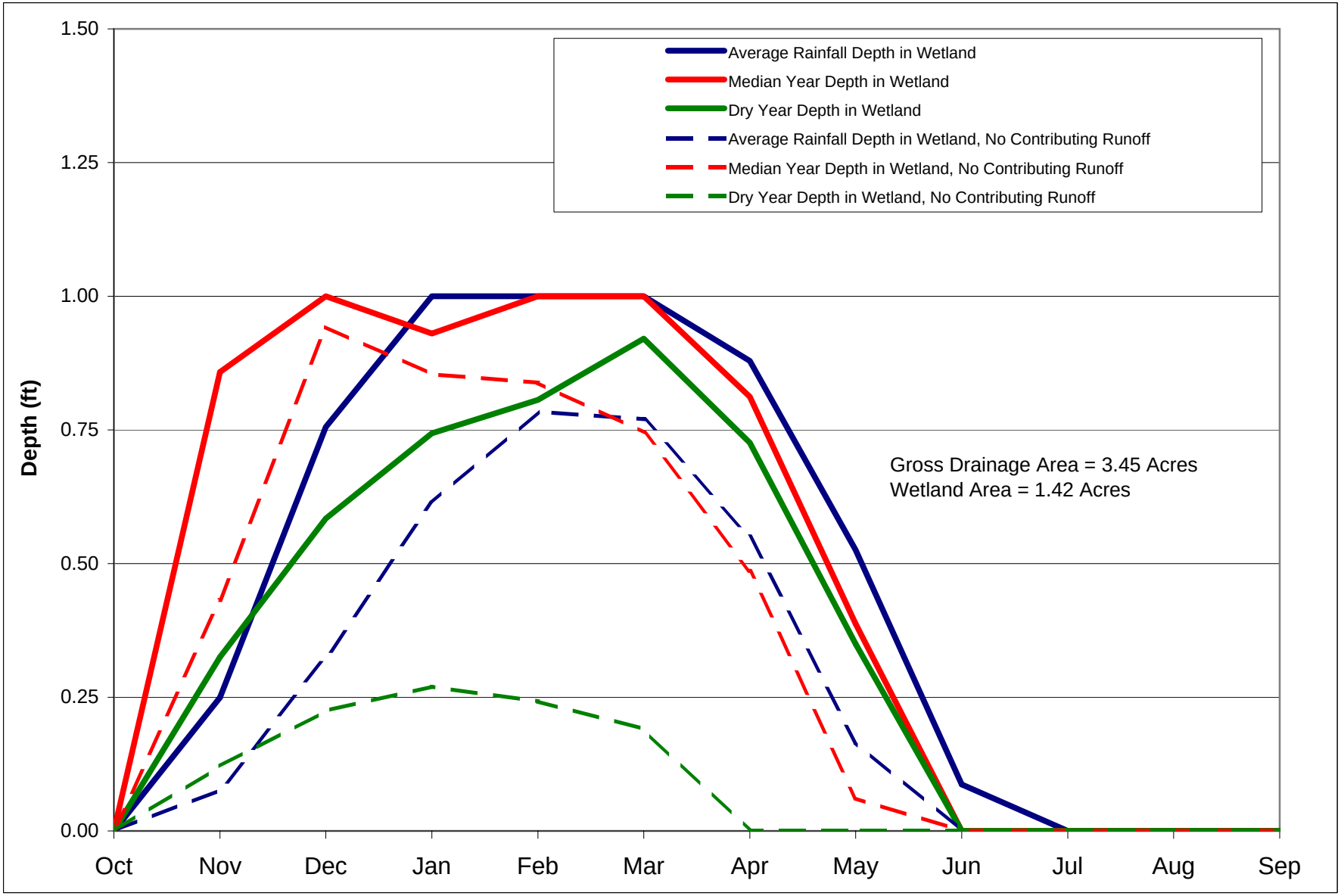

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Water Budget Results

Watershed D + Wetland 4

FIGURE 4D

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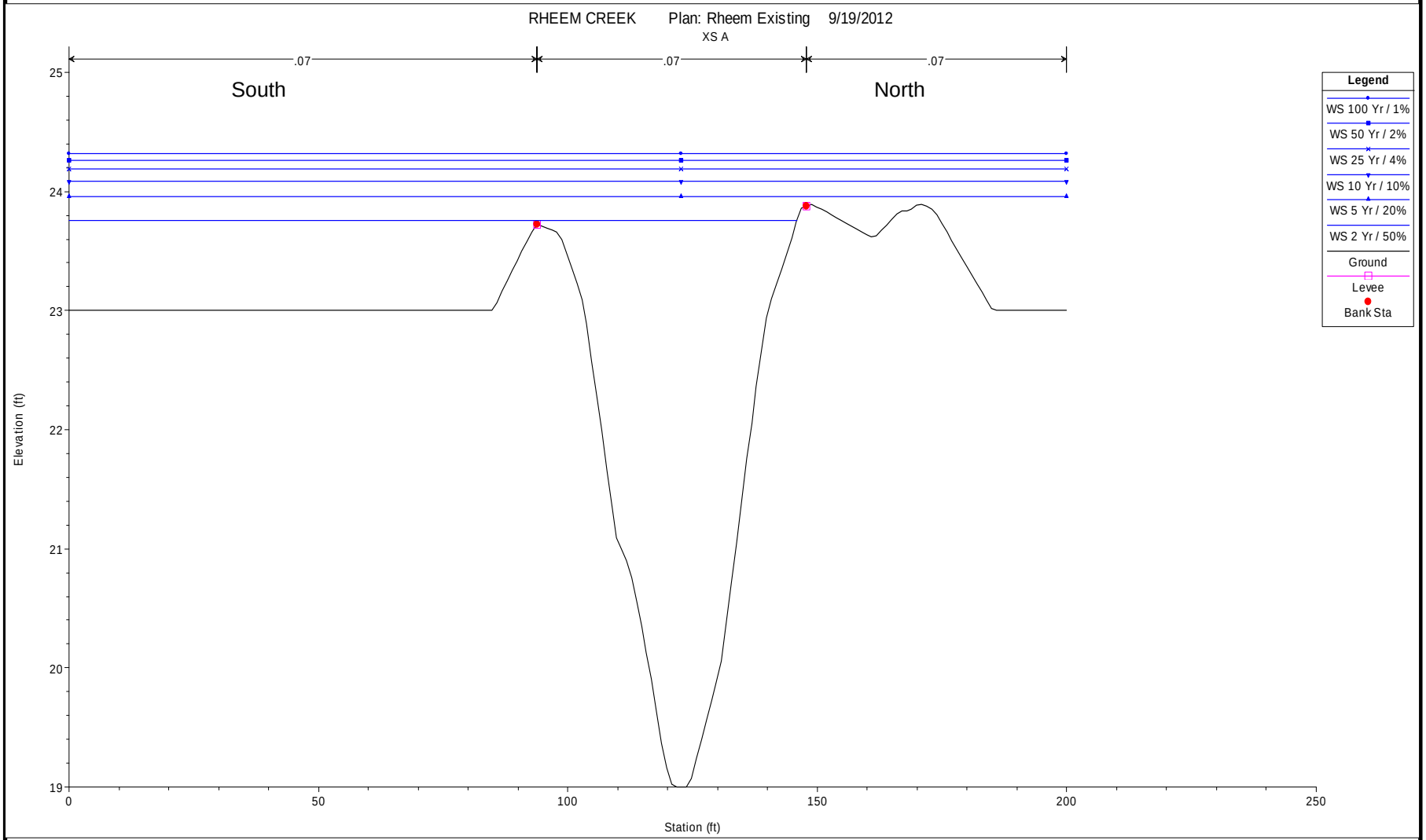

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Rheem Creek Cross-Section, Facing Downstream

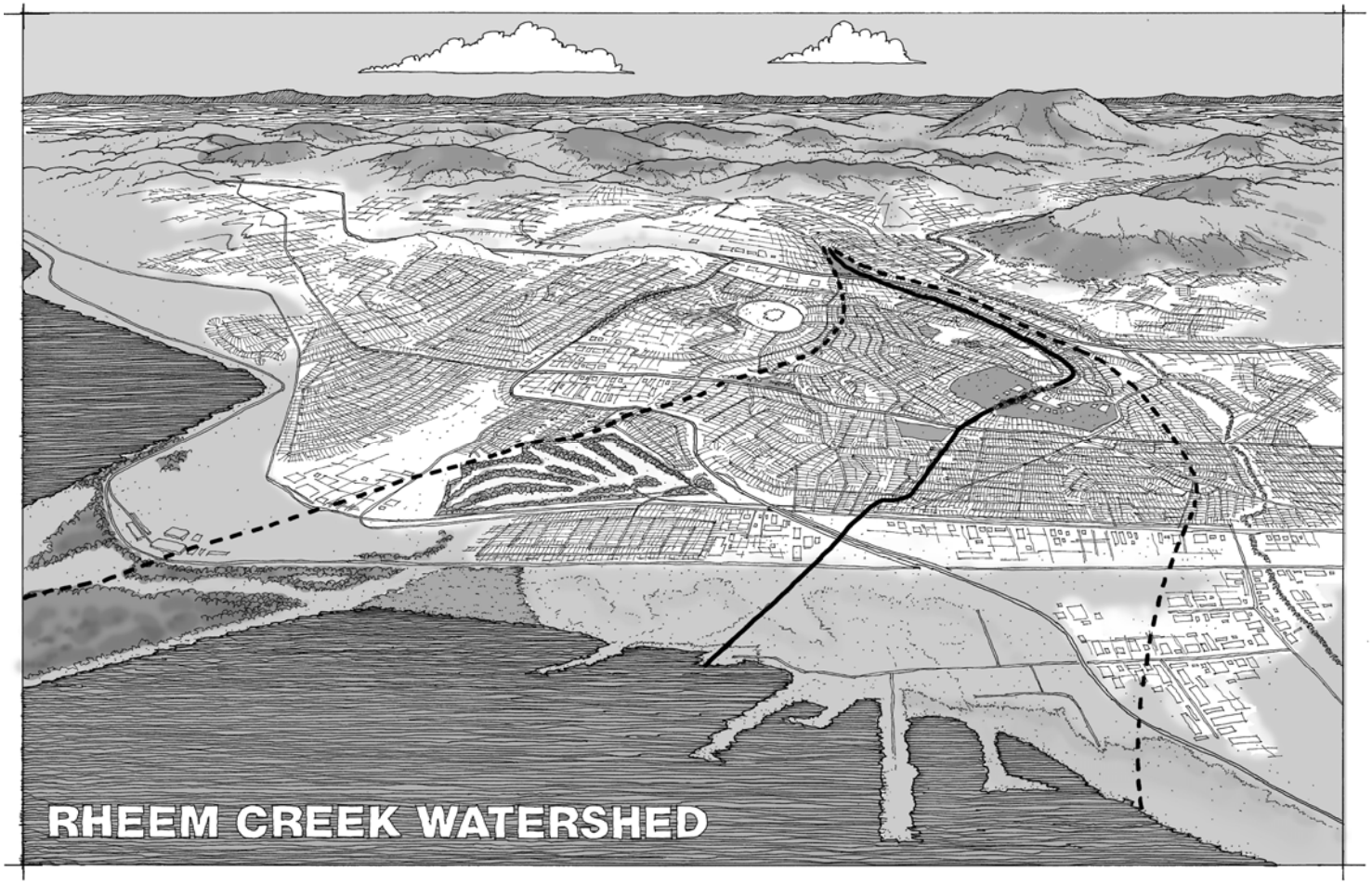
225 FT Downstream from Giant Road

FIGURE

5

Z:\3112_Olberding\7112 Rheem-Cr\Water Budget\Richmond Precip and ETo

RHEEM CREEK WATERSHED
ASSESSMENT AND CONCEPTUAL RESTORATION PLAN



PREPARED BY THE NATURAL HERITAGE INSTITUTE
FUNDED BY THE CALFED WATERSHED PROGRAM
MARCH 2007

RHEEM CREEK WATERSHED
ASSESSMENT AND CONCEPTUAL RESTORATION PLAN

***We**, the community members and stakeholders of the Rheem Creek Watershed declare that we:*

1. *Are dedicated to the preservation of open space and ecosystems (where appropriate), for recreational, educational, and non-human uses throughout the Rheem Creek watershed.*

2. *Are committed to working on the restoration of natural services in our watershed, including Rheem Creek and its shoreline, because we recognize interconnectedness between the well being of our environment and the well being of our communities.*

3. *Are dedicated to involving and working alongside a diverse team of stewards, promoting teamwork and collaboration among community members and allies from all parts of the watershed, and of all ages, backgrounds, and ethnicities.*

4. *Are devoted to the idea that the health of our watershed and the right to a clean environment are matters of environmental justice for our communities.*

The work and dedication of each and every one of us matters.

-The Rheem Creek Declaration, August 5, 2006

ACKNOWLEDGEMENTS

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Maps
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Funded by
CALFED Watershed Program

Project Partners
Community Health Initiative
Parchester Village Neighborhood Council
Urban Creeks Council
West County Toxics Coalition

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INTRODUCTION

The Natural Heritage Institute completed this report as part of a CALFED Watershed Program-funded project focused on the Rheem Creek watershed. The CALFED Watershed Program funded the research, writing, and printing of this report, community visioning activities, and watershed monitoring between 2004 and 2007.

This report provides an overview of the ecological and social history and conditions of Rheem Creek and its watershed lands. The intention of the report is to draw attention to Rheem Creek, lay groundwork, and help residents gain an appreciation for what is at stake in the watershed and how they can get involved.

Part One provides the background, describing how geological processes, climate, soils, topography, and tides have all interacted to form the Rheem Creek watershed. Part Two tells the story of human residents, describing the development of the watershed and the coincident loss of natural habitats. Part Three relates human activities – notably development – to the amount, timing, and quality of water in the creek channel.

Part Four describes the community visioning process that helped to develop the conceptual restoration plan.

Part Five presents a number of enhancement and restoration opportunities that address the critical issues identified by this assessment and by the community visioning process. We are hopeful that this report can lead the way to a healthier watershed for both human and non-human residents.

I. SETTING AND BACKGROUND

The information presented in this chapter sets the stage for understanding Rheem Creek and its watershed. Geological processes, climate, soils, topography, and tides have all interacted to form the Rheem Creek watershed as humans have known it for over 10,000 years.

The Rheem Creek watershed drains 2.8 square miles of land in western Contra Costa County, California, about 20 miles northeast of San Francisco. Figure 1 shows the location of the watershed within the San Francisco Bay Area. The Rheem Creek watershed contains urban land and open space within the cities of Richmond and San Pablo as well as the unincorporated communities of Rollingwood and North Richmond (Contra Costa County Community Development Department and the Contra Costa Watershed Forum 2003). Figure 2 depicts the cities, major roads, and important landmarks within the watershed. While it is the smallest watershed within Contra Costa County, draining just 0.4% of land in the county, the undeveloped bayfront land is an important ecological link in the baylands ecosystem (the lands within the boundaries of the tides) which has been severely affected by human development patterns.

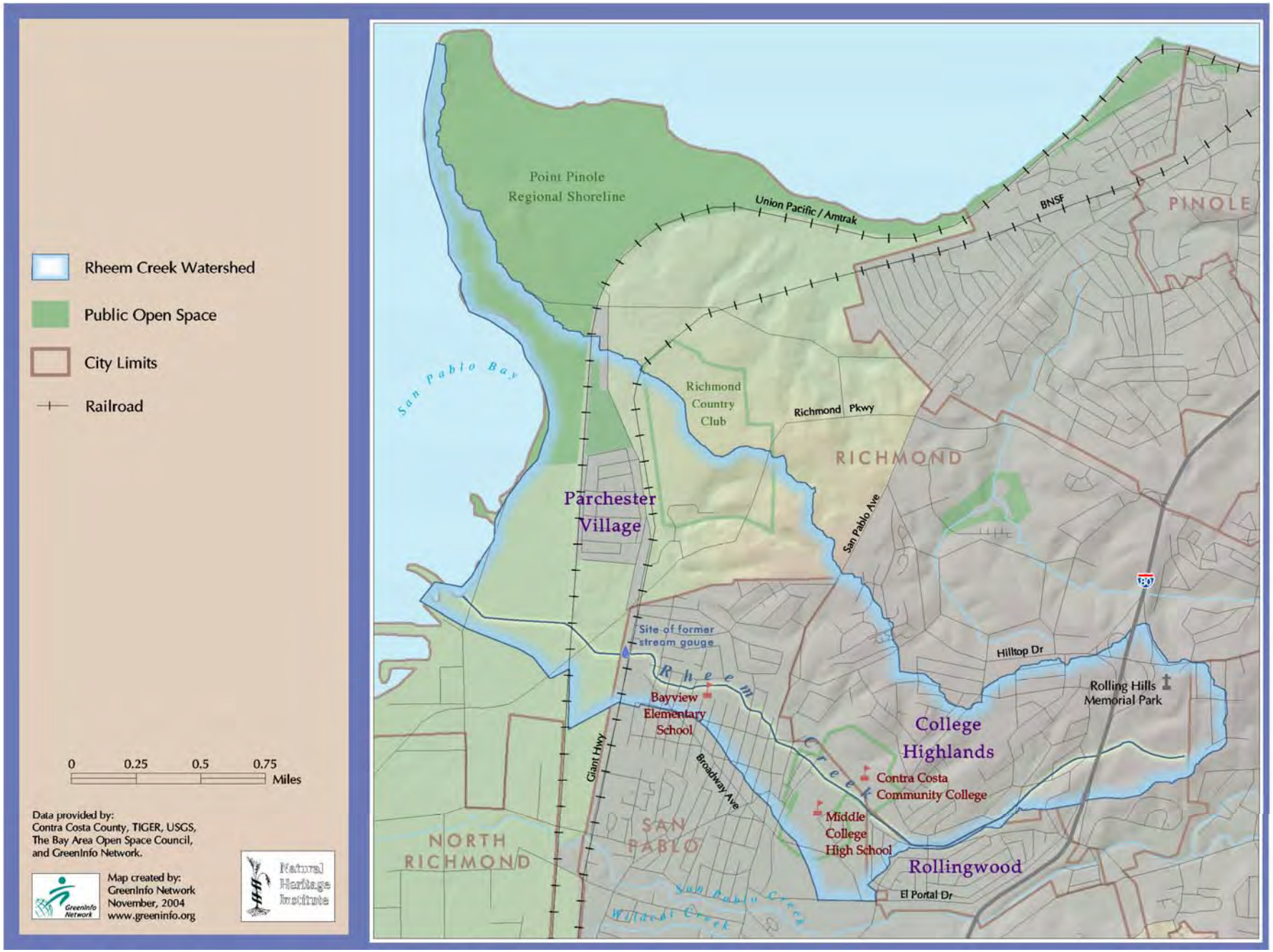
Rheem Creek flows westward for about 3.4 miles from its headwaters in the East Bay Hills east of Interstate 80 to its mouth at the San Pablo Bay, one-half mile south of Point Pinole. In its course toward the Bay, Rheem Creek passes through a diverse mix of land uses, including multiple residential neighborhoods, a cemetery, an industrial area, a community college, a schoolyard, and a large undeveloped section of land. The watershed lands include a country club, several neighborhood parks, and a section of the Point Pinole Regional Shoreline.

Regional Setting

The small Rheem Creek watershed is nested within two much larger watershed systems: the San Pablo Bay watershed and the San Francisco Bay-Delta Estuary watershed (see Figure 3). With its mouth at the San Pablo Bay, Rheem Creek is a drainage of the San Pablo Bay watershed, which encompasses about 900 square miles within Napa, Sonoma, Solano, Marin, Contra Costa, and Alameda Counties (San Pablo Bay Watershed Restoration Program 2000). This watershed is made up of all the watershed lands of the streams that flow into the San Pablo Bay, including Rheem Creek, located in the southwestern part of the watershed.

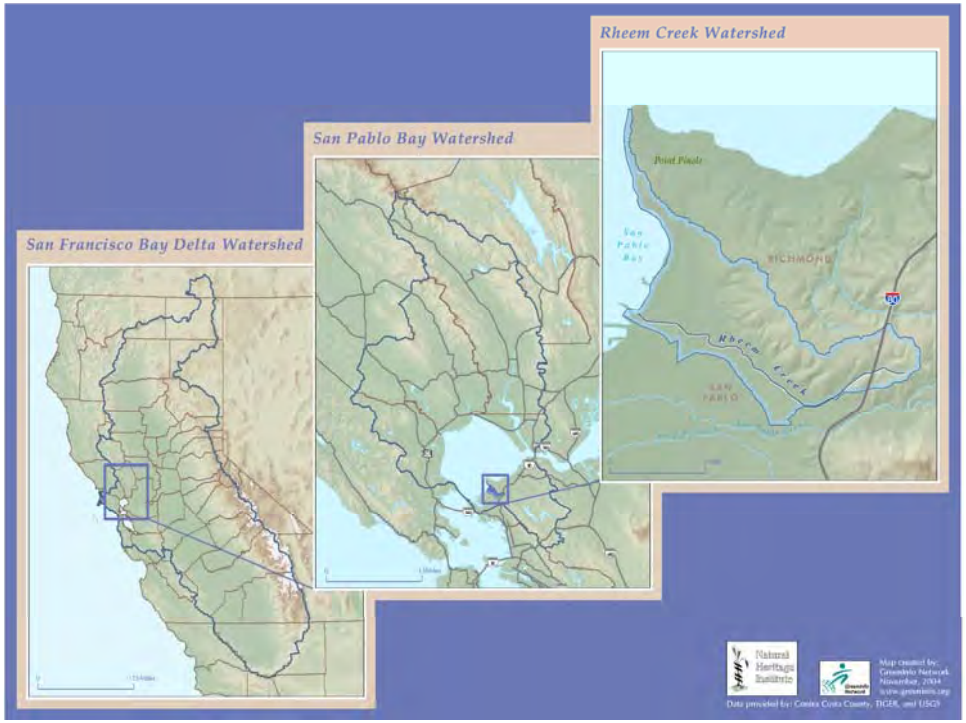
Figure 1: Rheem Creek Watershed Location Map

Figure 2: Rheem Creek Watershed Political Map



The San Pablo Bay watershed is a watershed within the still-larger San Francisco Bay-Delta watershed. San Pablo Bay is one of four large embayments within the greater San Francisco Bay, which is the downstream part of the San Francisco Bay-Delta Estuary. The Bay-Delta watershed contains 61,000 square miles of land, 42% of California's land area (The Bay Institute 1998). This huge watershed includes the western slopes of the Sierra Nevada and Cascade ranges, the Central Valley, and much of the San Francisco Bay Area, including the Rheem Creek watershed.

Figure 3: Watershed Series Map



An estuary is an area where fresh water from rivers and streams mixes with salt water from the sea. Freshwater from rivers and streams flows into the San Francisco Bay-Delta Estuary by gravity, while saline water from the Pacific Ocean is pushed inland by tidal action. The San Francisco Bay-Delta Estuary is the largest estuary on the west coast of North America. Although most of the freshwater entering the estuary flows from the western slopes of the Sierra Nevada mountain range and enters the estuary via the large Sacramento and San Joaquin rivers, the streams that drain to the Greater San Francisco Bay, such as Rheem Creek, also contribute freshwater to the estuarine system.

What's a Watershed?

Everyone lives in a watershed. A watershed is all of the land area that drains into a particular creek, river, lake, ocean, or other body of water. The watershed includes the landforms, vegetation, habitat, biological systems and natural communities contained within its boundaries, as well as the water body itself. Healthy watersheds benefit the plants, animals, and people that live within their boundaries.

Watershed management is the coordination of the numerous activities that affect the natural resources and health of the watershed and the body of water. Watershed management activities include flood protection, water conservation, preserving and creating open space for recreation and habitat for land and water animals, and reducing pollution of surface water resources. In California and around the world, public agencies, environmental groups, and community organizations have increasingly come to recognize the importance of managing natural resources at the watershed level.

Sections of the Watershed

Despite its small size, the Rheem Creek watershed has four distinct sections, distinguished by their different topography, geology, former natural habitats, and present land uses. These sections are identified on Figure 4. Greater detail about the watershed land and stream channel within each section is provided in subsequent chapters.

Headwaters: The creek's headwaters are located on the western edge of the East Bay Hills, in what is now a residential neighborhood of El Sobrante. This section of the watershed is characterized by fairly steep terrain. The creek is first visible where it flows through the Rolling Hills Memorial Park in El Sobrante east of Interstate 80.

Figure 4: Sections of the Rheem Creek Watershed

Upper watershed: The upper watershed is bounded by Interstate 80 to the east and San Pablo Avenue to the west. The upper watershed is an area of gentle hills that includes two residential neighborhoods: the College Highlands subdivision in Richmond and the unincorporated Rollingwood neighborhood. This section of the watershed also contains Contra Costa College in the City of San Pablo.

Middle watershed: The middle of the watershed, bounded by San Pablo Avenue to the east and Giant Highway to the west, is a flatter and densely developed section. It includes the neighborhoods in the northwestern part of the city of San Pablo and an industrial area along Giant Highway.

Lower watershed: This is the flattest and least developed section of the watershed, bounded by Giant Highway to the east and the San Pablo Bay to the west. The only residential community in the lower watershed is Parchester Village in the City of Richmond. Rheem Creek meets the San Pablo Bay at the Breuner Marsh, just north of the Richmond Rod and Gun Club and south of Giant Marsh. The land in this section of the watershed is in North Richmond, an unincorporated section of Contra Costa County.

The map in Figure 4 includes in the lower watershed lands that drain directly to the San Pablo Bay, not to the Rheem Creek channel and thus are technically not within the watershed. However, we have included these lands as part of the Rheem Creek watershed because of their ecological connectivity with the tidal baylands just to the north of the channel. As discussed later in this report, historical maps indicate that small marsh channels north of Rheem Creek formerly drained directly to the San Pablo Bay.

Near the mouth of Rheem Creek are remnants of tidal flat and tidal marsh habitat, two habitat types that have been severely affected by human development all around the San Francisco Bay area. These habitats are important to hundreds of fish and wildlife species, including many that are in danger of extinction, as well as the 6.8 million people living in the

Bay Area (Goals Project 1999). Because the land remains mostly undeveloped and because of its ecological importance, the lower watershed has great potential for protection and restoration. Efforts to protect and restore the lower watershed are already underway. These efforts are also unique opportunities for the involvement of local residents in watershed enhancement and education.

Watershed Formation

Geographically, the Rheem Creek watershed is situated within the northern Coastal Range of California. The East Bay hills, where Rheem Creek's headwaters are located, are a part of this mountain range. The Coastal Range formed about three million years ago. As the North American continent moved westward and the Pacific plate slid underneath it, there was uplifting of ocean sediments and volcanic rocks. This uplifting ultimately created the Coastal Range (San Pablo Bay Watershed Restoration Program 2000).

This plate movement occurs along multiple faults, including the San Andreas Fault and the Hayward Fault (San Pablo Bay Watershed Restoration Program 2000). As shown in Figure 5, the Northern Hayward Fault runs through the Rheem Creek watershed, crossing Contra Costa College in the middle watershed and passing through Point Pinole Regional Shoreline at the northwestern edge of the watershed.

The San Francisco Bay itself formed about 10,000 years ago during a period of glacial melting. As glaciers around the world melted, sea level rose, and the Pacific Ocean moved eastward. The San Francisco Bay changed from a dry valley with a river to a "drowned" river valley, inundated by the sea. Suspended, sediment was deposited along the margins of San Pablo Bay, becoming mudflats and marshes (San Pablo Bay Watershed Restoration Program 2000).

Climate

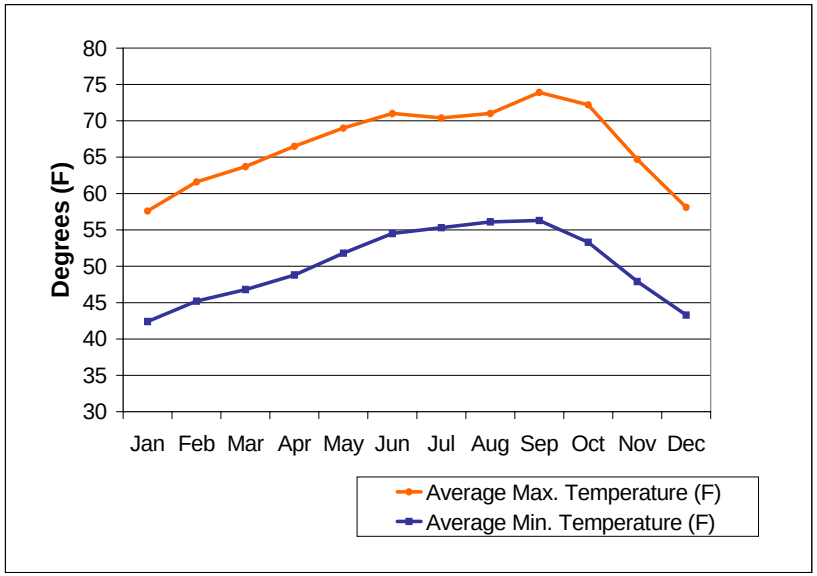
Like most of the San Francisco Bay Area, the Rheem Creek watershed has a Mediterranean climate, which is characterized by cool, moist to wet winters and cool to warm, dry summers (San Pablo Bay Watershed

Figure 5: Faulting in the Rheem Creek Watershed

Restoration Program, 2000). The proximity of the Pacific Ocean has a moderating effect on the climate compared to more inland parts of the San Francisco Bay Area. While night and morning fog from the ocean is common at all times of year, the frequent presence of summer fog leads to lower average summer temperatures than in areas further east in Contra Costa County.

The nearest National Weather Service station is located at Richmond, three miles south of Rheem Creek. Data for this station, which has been in operation since 1950, is distributed by the Western Regional Climate Center. The mean annual temperature for the City of Richmond is 58.4° F. Because the entire Rheem Creek watershed is at low elevation and in close proximity to the Bay, winter temperatures rarely drop below freezing and snowfall has never been recorded at the Richmond weather station. The average winter (December to February) temperature in Richmond is 51.4° F, with the coldest temperatures typically occurring during January, when the average low is 42.5° F. The average summer (June to August) temperature in Richmond is 63.1° F. The warmest temperatures have been recorded in September, with an average high of 73.9° F. Figure 5 indicates the average temperature in Richmond throughout the year.

Figure 5: Average Monthly Temperature in Richmond, CA (1950-2003)



Annual rainfall varies considerably from year to year. Although the average annual rainfall in Richmond is 22.5 inches, "average" years are rare: only four years in the 52-year record (1951-2003) had rainfall between 22 and 23 inches. This variability is illustrated in Figure 7. In 1994, the year of lowest annual rainfall, only 8.97 inches of rain fell in Richmond. In contrast, in 1983, the year of highest rainfall, 47.49 inches of rain were recorded. This was more than five times what fell in 1994.

Despite the extreme variability in the amount of rainfall from year to year, the timing of rainfall is quite predictable. Over 90% of annual rainfall typically falls between the months of November and April. As seen in Figure 8, the highest monthly mean precipitation occurs in December, January, and February, and little if any rainfall in the months of June, July, and August.

Figure 6: Monthly Average Rainfall in Richmond, CA (1951-2003)

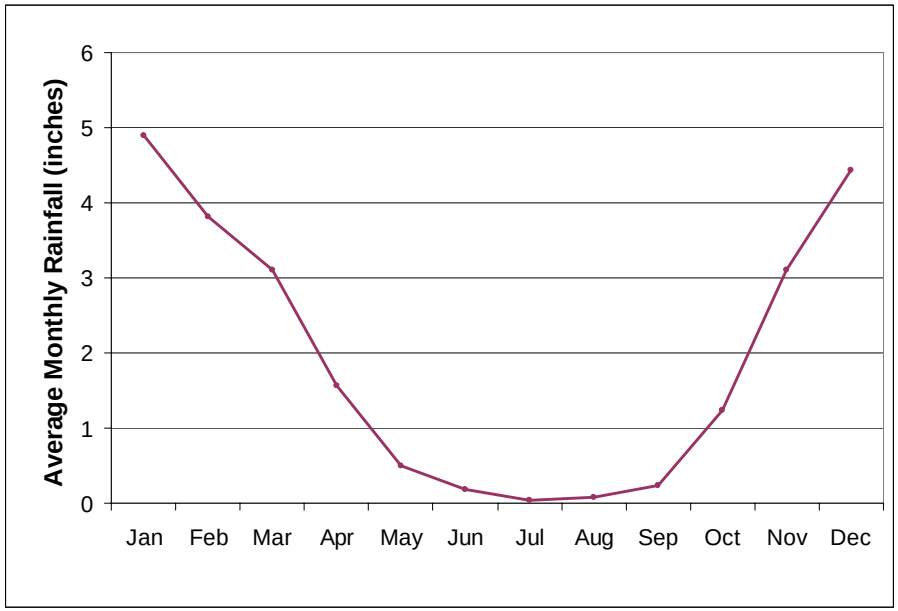
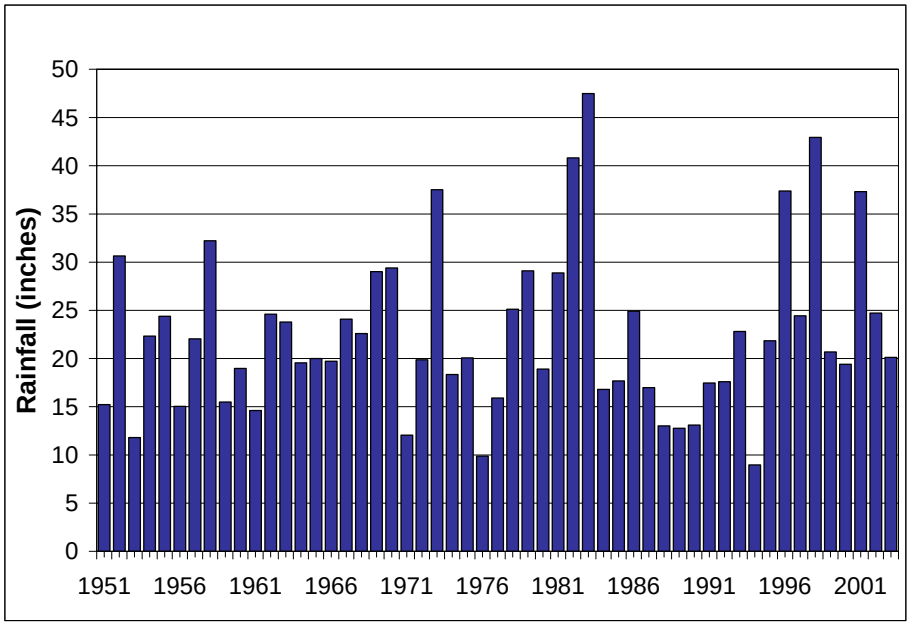


Figure 7: Total Annual Rainfall in Richmond, CA (1951-2003)



Soils

According to the United States Department of Agriculture, Soil Conservation Service (1977), all of the soils in the Rheem Creek watershed are of the Tierra-Antioch-Perkins Association. These soils are found on land that is nearly flat to moderately steep, and the soils themselves consist of moderately well drained to well-drained loams and clay loams. These soils formed in old alluvium – the unconsolidated material deposited by running water such as streams – on nearly level shelves of land. In addition, soils adjacent to the bay formed from sediment that had been suspended in ocean water.

In the headwaters of the watershed, soils are mapped as Los Osos clay loam, a hilly soil on upland areas with slopes of 15-30%. These are well-drained soils underlain by sandstone and shale, where the erosion hazard is moderate when soils are bare.

In the upper and middle sections of the watershed, between Interstate 80 and Giant Highway, soil types are a combination of Cut and Fill and Tierra loam. Cut and Fill soils are the result of extensive human manipulation of sloping land (with a slope between 9-30%) for urban use, notably residential development. In these areas, soils have very slow to slow permeability, runoff occurs rapidly, and the erosion hazard is high. In contrast, Tierra loam soils, formed from material weathered from deposits on sedimentary terraces, are well drained, and have lower levels of runoff and a lower likelihood of erosion. However, the erosion hazard can be high when soils are bare (i.e. lacking vegetation).

In the lower watershed, soils are primarily Clear Lake clay and Reyes Silty clay. Clear Lake clay soils are poorly drained soils typically found within coastal valleys. These clayey soils remain moist throughout the rainy season. Runoff occurs very slowly, and thus there is no hazard of erosion. Typical vegetation includes annual grasses and forbs. Reyes Silty clay soils are found at the mouth of the creek, adjacent to the San Pablo Bay. These are very poorly drained soils that always remain moist and are inundated at high tide. The water table is close to the surface underneath these soils, which are characteristic of saltwater marshes affected by tidal action. Typical vegetation includes pickleweed, saltgrass, and sedges.

Topography

Rheem Creek descends about 200 vertical feet over its 3.4 mile journey westward from the East Bay Hills to the San Pablo Bay. Its average slope over its course is about 1%. In its uppermost reach, the creek descends fairly steeply, with a gradient of about 4%, losing half of its elevation in its first half-mile. As it descends through the upper watershed, the channel becomes less steep, with a slope of between 0.5 and 1%. For the final 1.4 miles, the channel has a fairly consistent and very gentle slope of about 0.7%. Figure 9 shows an elevation profile of the creek.

Tidal Influence

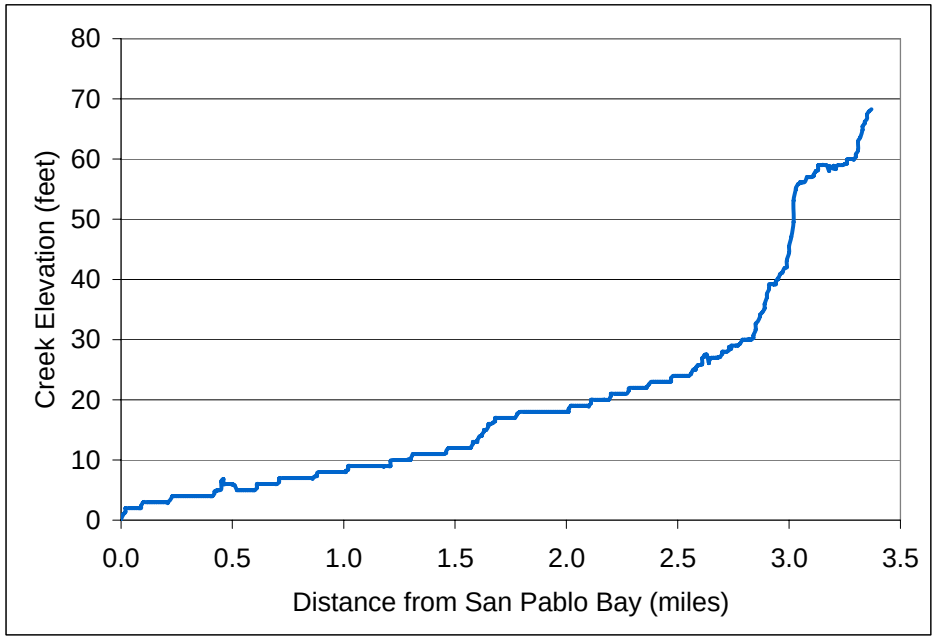
Tidal inflows from the Pacific Ocean affect the lower reaches of Rheem Creek. Each day, the San Francisco Bay has two high tides and two low tides. As the tide comes in, salt water from the Pacific Ocean passes

through the Golden Gate and flows through the Central San Francisco Bay, into the San Pablo Bay, and upstream into Rheem Creek and other tributaries of the San Pablo Bay.

Tidal processes dominate in the lowest quarter mile of Rheem Creek (Urban Creeks Council of California and FarWest Restoration Engineering 2002). This means that water in the lowermost reach of the creek is brackish; that is, this water is a combination of freshwater moving out toward the bay and saline water pushed inland by tides in the bay.

During a four-week cycle, the moon's phases influence the extremes of tides. "Mean" tides are the average water level over the four-week tidal cycle. The mean higher high tide at Breuner Marsh, adjacent to the mouth of Rheem Creek is 3.52 feet high, while the mean low tide is -1.68 feet (Urban Creeks Council of California and FarWest Restoration Engineering 2002). Over a four-week lunar cycle, the difference in height between the mean low and high tides at the mouth of Rheem Creek is 5.2 feet.

Figure 8: Elevation Profile of Rheem Creek



II. THE WATERSHED LANDS

Human activities have altered the Rheem Creek watershed lands in many ways. Since the arrival of Europeans to the San Francisco Bay Area in the late eighteenth century, the watershed lands have been affected by agriculture and ranching, the building of factories, and the construction of dense housing developments. This chapter presents a snapshot of watershed lands today: the land use patterns, human community, and remaining natural habitats. We also describe qualitatively how the changing human community has altered the watershed lands.

Watershed Land Uses

While the Rheem Creek watershed lands have undergone major changes in the past two centuries, it is likely that they will remain in their current state into the future. Most of the Rheem Creek watershed is now fully developed: streets, houses, and industry dominate the landscape (Figure 9). Other than the lower watershed, there are few places where new roads and buildings can be built. Today, about 50% of the land in the watershed consists of impervious surfaces, such as buildings, paved roads, and parking lots. Rainwater falling on these surfaces runs off the land directly into storm drains or creek channels rather than infiltrating into the soil. In contrast, other watersheds in the county have an average of 35% impervious surfaces (Contra Costa County Community Development Department and the Contra Costa Watershed Forum, 2003).

A higher percentage of impervious surfaces has many effects on the health of a watershed. For example, with more impervious surfaces, there are fewer places where plants can grow, and therefore less food for wildlife all the way up the food chain. The many effects of impervious surfaces on the creek channel itself are discussed in the next chapter.

By looking at the General Plans of the cities and counties that have jurisdiction over land in the watershed, we can gain a quantitative understanding of the composition of land uses. Because there are few undeveloped areas in the Rheem Creek watershed and because much of the remaining undeveloped land in the lower watershed is designated as open space, in the Rheem Creek watershed the "planned land uses" are roughly equivalent to actual land uses. (This is not the case in watersheds where development is still proceeding.) These planned land uses in the Rheem Creek watershed are depicted in, and the percentages of each planned land use are summarized in Table 1.

Table 1: Planned Land Uses in the Rheem Creek Watershed

Land planned for residential uses	42.1%
Land planned for open space and parks	22.2%
Land planned for public uses	22.2%
Land planned for industrial uses	7.2%
Land planned for mixed uses	3.5%
Water	2.3%
Land planned for commercial uses	0.4%
Land planned for agricultural uses	0.0%

In the Rheem Creek watershed, over 40% of the land is residential. Of this, 80% is occupied by single-family homes. About 22% of watershed lands have public and semi-public uses; this includes Contra Costa College, major roads such as Interstate 80 and the Richmond Parkway as well as smaller roads, and three public schools within the watershed (Contra Costa County Community Development Department and the Contra Costa Watershed Forum, 2003). About 7% of land is designated for industrial uses, while less than 1% of land area is designated for commercial uses. Most of the industrial land is in the lower watershed and remains largely undeveloped.

Land planned as open space or parks constitutes 22% of the total watershed, compared with the Contra Costa County average of 31%. With the exception of a few small parks in the upper and middle sections of the watershed, all of the open space and parkland in the watershed is in the lower watershed, which includes the Richmond Country Club, a section of the Point Pinole Regional Shoreline, and adjacent undeveloped bay- front land west of Giant Highway (Contra Costa Community Development Department and Contra Costa Watershed Forum 2003).

Figure 9: Aerial Photograph of Rheem Creek Watershed

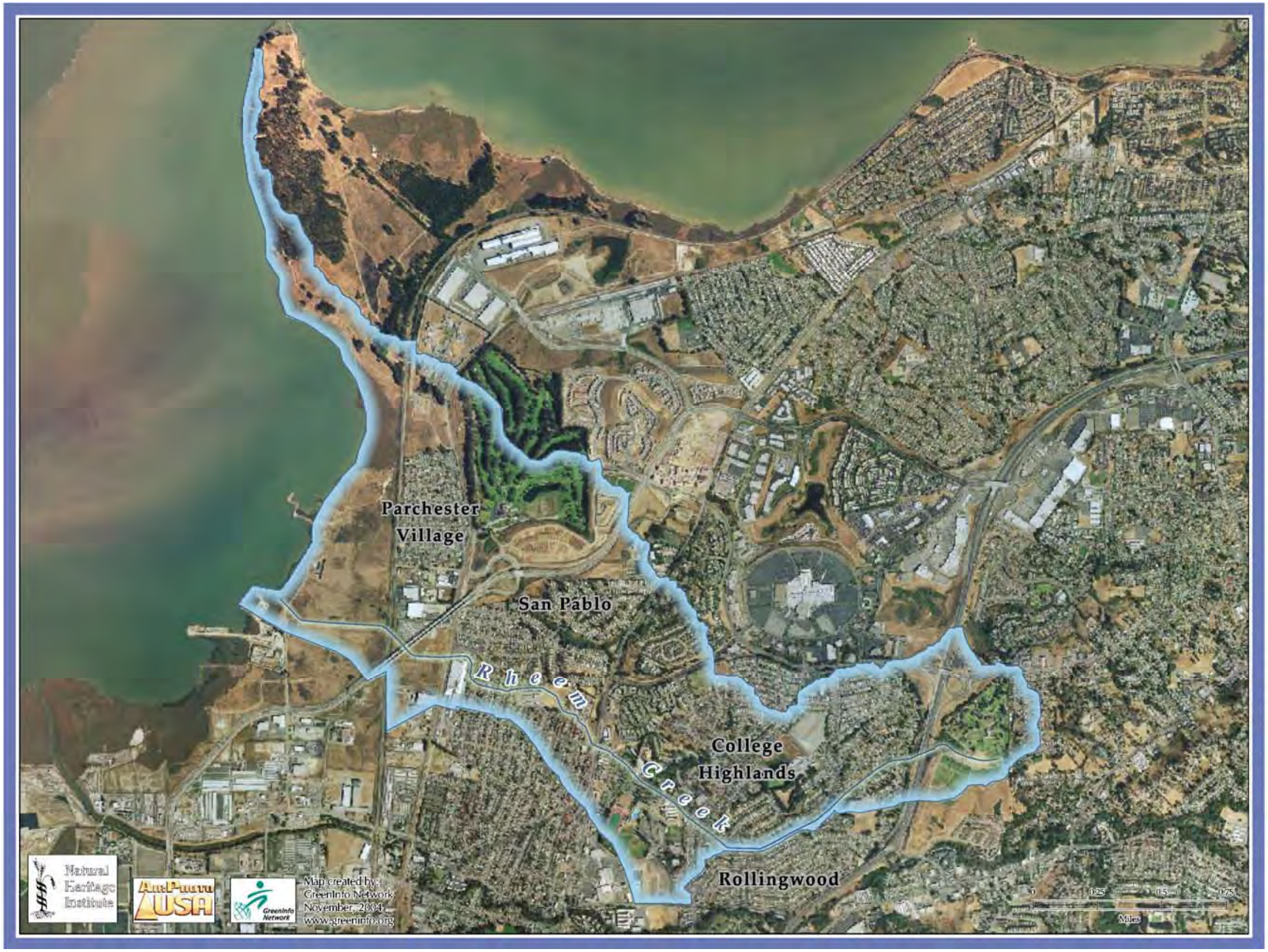


Figure 10: Planned Land Uses in the Rheem Creek Watershed

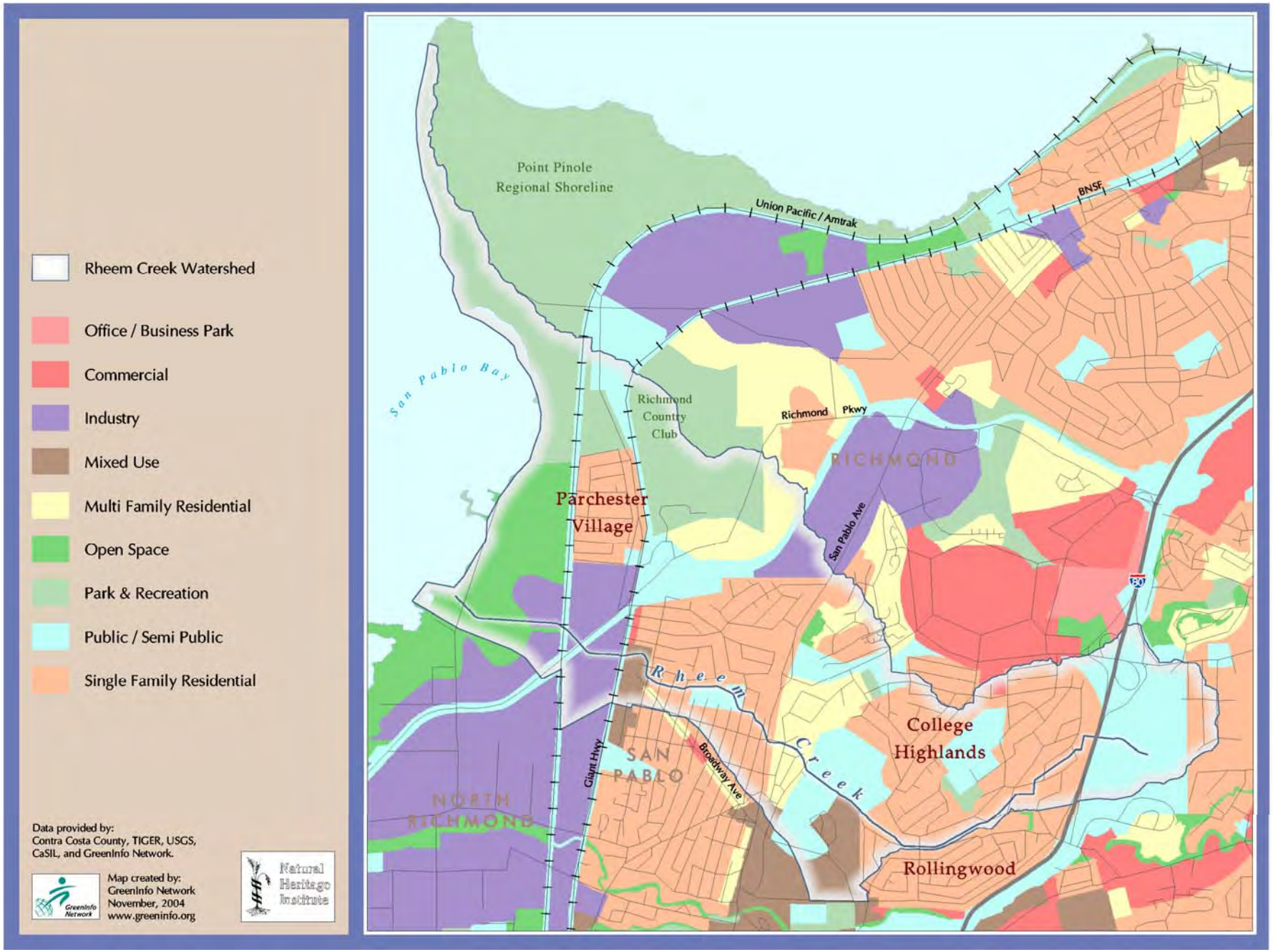


Figure 11: Headwaters

Headwaters: The upper watershed contains a combination of residential development on the edge of El Sobrante and open space contained within the Rolling Hills Memorial Park. Figure 12 shows the Rolling Hills Memorial Park, with the Rheem Creek channel covered in trees and shrubs, at the left.

Figure 12: Upper Watershed

Upper watershed: This section of the watershed includes two residential communities - the College Highlands subdivision in Richmond and the unincorporated Rollingwood neighborhood – and Contra Costa College in the City of San Pablo. The residential communities (Figure 12) are densely populated and entirely built out. In contrast, the College has a much lower density of buildings and maintains some open space.

Figure 13: Middle Watershed

Middle watershed: This section of the watershed, bounded by San Pablo Avenue to the east and Giant Highway to the west, includes the West San Pablo residential neighborhood and a region of industrial development at Giant Highway. This part of the watershed is densely developed, mostly with single family homes (Figure 14).

Figure 14: Lower Watershed

Lower watershed: This is the least developed section of the watershed. It is bounded by Giant Highway to the east and the San Pablo Bay to the west. While this section of the watershed is largely undeveloped, the land is heavily impacted by previous land uses, especially grazing. Non-native grasses grow here, although there are remnant patches of native tidal marsh and seasonal wetlands near the mouth of the creek (Figure 14).

The Human Community

The Rheem Creek watershed is home to a diverse human community. People live in various residential areas, including the College Highlands and Rollingwood neighborhood in the upper watershed, northern San Pablo in the middle watershed, and Parchester Village in the lower watershed. Our understanding of the composition of the human community comes from the most recent U.S. Census. Because census boundaries do not align perfectly with watershed boundaries, the information presented here is approximate.

About 12,700 people live in the Rheem Creek watershed, 1.3% of the total population of Contra Costa County. Ethnically, the watershed is extremely diverse; no single racial or ethnic group forms the majority of the population. 39% of people living in the watershed are African American, 26% are Latino, 16% are Asian, 14% are white, 3% are biracial, and 1% are American Indian. There is also significant diversity in terms of languages spoken in the household: 61% of households in the watershed speak English as their primary language, while 20% speak Spanish and 16% speak an Asian or Pacific Island language. Tables 2 and 3 compare the ethnic composition and languages spoken in the watershed with those of Contra Costa County, which is less diverse in racial and linguistic terms. In the county, white people constitute a majority of the population and a higher percentage of households are English-speaking.

Table 2: Racial and Ethnic Composition of the Rheem Creek Watershed

	% Rheem Creek Watershed Population	% Contra Costa County Population
Black or African American	39%	9%
Hispanic or Latino	26%	18%
Asian	16%	11%
White	14%	58%
Two or more races	3%	4%
American Indian and Alaska Native	1%	0%

Table 3: Languages Spoken in the Rheem Creek Watershed

	% Rheem Creek Watershed Households	% Contra Costa County Households
English	61%	73%
Spanish	20%	12%
Asian or Paficic Island language	13%	7%
Other	6%	8%

The average household size in the watershed is 3.3 people, compared with Contra Costa County's average household size of 2.8. In the watershed and the larger county, about 70% of the households are comprised of families living together, and about half of these families have children under 18. There is a larger proportion of single family households in the watershed than in the County: 62% are married couple households, while 38% of the family households in the watershed are single parent families, compared with 21% in the Contra Costa County.

Average income in the Rheem Creek watershed is lower than in Contra Costa County. The median family income in the watershed is $50,000, just 69% of the median household family in the county ($73,000), and lower than the median of $53,000 for all of California. The poverty rate in the watershed is also much higher: 15% in the watershed, compared to 8% in Contra Costa County (and 10% in California). While incomes in the watershed are lower than elsewhere in the county, rents are not much lower: the median contract rent in 2000 was $770 in the watershed, compared with $826 in the county. However, home values are substantially less in the watershed. In 2001, the median value of a home in the watershed was $153,000, which is 60% of the county's median home value of $254,000. Homes in the watershed are generally older and smaller than those in the rest of the county.

Educational achievement in the watershed is lower than in the larger county. In terms of education, 28% of people 25 years or older in the Rheem Creek watershed have not finished high school, compared with 13% in Contra Costa County. Just 21% of people aged 25 or older in the

watershed have attained an associate, bachelor's, master's, professional school, or doctoral degree, compared with 43% of people in Contra Costa County.

We have seen that the community of the Rheem Creek watershed is quite different than that of other watersheds in Contra Costa County. The population is more diverse, less affluent, and less educated. Residents of the watershed also have less access to open space than residents of other watersheds in the county. No single factor is sufficient to explain why these disparities exist. However, a consideration of the history of the watershed can help us to appreciate how the human community has changed and evolved over time.

The History of Human Settlement in the Watershed

The Rheem Creek watershed has been inhabited by humans for over ten thousand years. For most of this period, it was sparsely populated. Prior to the arrival of the Spanish in the Bay Area, the watershed was inhabited by different tribes of indigenous people who depended on the local resource base for their survival. With the arrival of Europeans in the late 1700s, the Rheem Creek watershed was transformed into an agricultural zone for raising livestock and growing crops such as wheat. By the 1900s, residential development was underway. The population of the area exploded during World War II, as thousands of people moved in to work in the shipyards. By the 1960s, the watershed was extensively urbanized. In the span of just two and a half decades, the human community completely transformed the watershed.

Earliest Inhabitants: The Huchiun

Records dating back to the late eighteenth century form a picture of the Rheem Creek watershed's human history. Our earliest knowledge about the lifestyle of the Native Americans who lived in the area comes from documents written by the Spanish after their arrival in the 1770s. By studying these records, Banks and Orlins (1981) compiled a detailed human history of the region. Unless otherwise noted, the historical information in the next two sections is based on their report.

The earliest settlements along the San Pablo Bay date back about 12,000 years (San Pablo Bay Watershed Restoration Program 2000). The group believed to have occupied all of western Contra Costa County, the land now within the cities of Richmond, San Pablo, and El Cerrito, were called the Huchiun people. Although the inland extent of land controlled by the Huchiun is unclear, it is believed that they controlled somewhere between 35 and 150 square miles.

For the Huchiun, the San Pablo Bay area offered abundant resources and provided for a sustainable and relatively stable existence. Along a 1.2 mile-long stretch of Rheem Creek, archaeologists identified four prehistoric sites, including one at the mouth (see Figure 15). The density of these sites indicates that the Rheem Creek environment was quite favorable for habitation by the Huchiun. An understanding of how the Huchiun lived provides insight into the diversity of locally available resources in the Rheem Creek watershed prior to the introduction of agriculture in the 1800s.

The Huchiun depended on local resources for their shelter, food, and transport. They lived in dome-shaped grass houses occupied by ten to twelve people. Population density was very low, about 2 to 4 people per square mile. It is believed that the population moved seasonally to avail themselves of different food sources. The sites found along the Bay may have been permanent villages occupied throughout the year, winter villages used in cold months, or temporary camps of people from other winter villages from nearby or more distant interior valleys.

The Huchiun had a hunting and gathering lifestyle. The staple foods of the Huchiun were plants, collected when seasonally available. Major foods included the greens of wild clover, Lupines, and Spindleroot, the roots and bulbs of Soap Plant, and the seeds of flowers such as California Buttercup, Chia, Farewell to Spring, and Red Maids. The acorn was also an important food source. Huchiun from the Rheem Creek area are believed to have consumed acorns from the coast live oak and to have gone inland to gather or trade acorns from other oak species such as white oak. Like acorns, nuts from the California Buckeye and California

Laurel were consumed, but these food sources all required lengthy preparation: drying, grinding, and leaching with water. The basket willow and the basket sedge were important non-food plant species used for weaving baskets.

Animal food sources included seafood, waterfowl, and mammals. They used boats made of tule reeds to fish for sturgeon, salmon, shellfish. Nets were used to capture waterfowl such as Canada geese, snow geese, American widgeon, pintail, mallard, green-winged teal, and shoveler. Some of the mammals in the Huchiun diet were deer, elk, and rabbit. Deer and elk were hunted with bows and obsidian-tipped arrows. The Huchiun also consumed antelope, sea otters, whales, sea lions, mice, California quail, robins, ground squirrels, and grasshoppers.

The Missions

The Huchiun hunting and gathering culture was destroyed after the arrival of the Spanish in the San Francisco Bay Area. The first recorded contact between the Huchiun and the Spanish occurred in 1772. Four years later, the Spanish founded Mission Dolores in San Francisco. Between 1776 and 1794, over 200 Huchiun were forcibly converted and baptized as Catholic and came to live at the Mission. Many Huchiun resisted the missionary's efforts, and in response, the Spanish sent military expeditions to the East Bay in the early 1800s. Their military force appears to have broken the resistance. Prior to European contact, the entire Huchiun population was most likely between 580 and 800 people; of these, 430 were baptized. Between 1803 and around 1815, former Huchiun lands were mostly uninhabited, although the Huchiun did cross the Bay regularly from San Francisco to camp and gather food.

The Huchiun who were converted and brought to the Mission fared very poorly. Infant mortality at the Mission was very high, and there were outbreaks of measles and dysentery. In one year (1806), over 300 people died of measles (Cole 1980). By 1822, only 49 of the 430 who had been baptized were still alive.

Rancho San Pablo

Finding that the ground in San Francisco was difficult to cultivate, priests from the Mission Dolores established an outstation along the San Pablo Bay to raise cattle and food for the mission around 1815. They called this outstation Rancho San Pablo. Ramón Abella, a priest from the mission who explored the bay area, had named the area San Pablo in 1811 after Saint Paul. Rancho San Pablo became the first permanent non-Indian settlement in what is now Contra Costa County.

After Mexico gained independence from Spain in 1821, the land that had been owned by the missions was divided up. In 1823, the Mexican government granted 18,000 acres of land to Francesco Castro and his family, and they came to live on the Rancho San Pablo. The Rheem Creek watershed was entirely encompassed within the Rancho. Francesco Castro had seen the hills of Contra Costa (Spanish for the opposite coast) from his home in the hills of San Francisco's Presidio, and he was the first white person to settle in Contra Costa (Cole 1980).

Through much of the nineteenth century, the Castros raised cattle, sheep, and horses and grew fruit, grain and vegetables at Rancho San Pablo (Cole 1980). The alluvial deposits of Rheem Creek were favorable for grazing livestock and growing crops (Urban Creeks Council and FarWest Engineering 2002). Cattle roamed freely on the hills and valleys. A ferry line across the bay connected East Bay ranchers to San Francisco markets. In addition, gold miners heading across the state purchased cattle, wheat, and potatoes from the Rancho before making their way east (Banks and Orlins 1981).

The Castro family forced many Native Americans to work on the Rancho San Pablo. Any descendants of the Huchiun converts who survived the Mission system were likely a part of this work force. The Rancho San Pablo was noted for its cruel treatment of Native Americans, and many who labored there died of starvation. Given the conditions at the Mission and on the Rancho, it is unknown whether any descendants of the Huchiun are alive today (Banks and Orlins 1981). In less than one hundred years, the Rheem Creek watershed was transformed from a

place where indigenous people lived on the land's natural resources to a place dominated by ranching and farming. In the span of two hundred years, the percentage of the watershed population that was American Indian dropped from 100% to just 1% today.

Industrial Development and Population Growth

In the middle of the nineteenth century, the Castro family began to sell off portions of the Rancho to American ranchers and farmers. In the lower Rheem Creek watershed, entrepreneurial farmers grew grain, operated warehouses, and grazed dairy cattle (Urban Creeks Council and FarWest Engineering 2002). By 1894, the Rancho San Pablo had been subdivided and claims to most of the land were granted to American settlers.

In the early 1900s, "agricultural hamlets with cultivated wheat fields were…transformed on subdivision maps to blocks of uniform city lands" (Banks and Orlins 1981). The Rheem Creek watershed developed more slowly than the surrounding area in the early twentieth century. At the turn of the century, only a few roads – Giant Highway, San Pablo Avenue, and a network of roads on Pinole Point – had been built in the watershed, and there were only a handful of buildings. By 1915, Broadway and adjoining streets in present-day San Pablo had been platted, but the only buildings in the watershed were those along Giant Highway and on Pinole Point.

Outside of the watershed, in the present-day Richmond area, the population increased over one hundred-fold, from 200 people in 1901, to 23,000 in 1923 (Cutting 1917, Richmond Chamber of Commerce 1944, cited in San Francisco Estuary Institute 2001). Industrial development accompanied the dramatic population expansion and real estate development. The two major industries at the turn of the century were the Atchison, Topeka, and Santa Fe Railroad and Standard Oil Company, which quickly became the west coast's largest refinery and one of the largest refineries in the world. In the late 1890s, the railroad completed construction of the last seventy miles of the transcontinental railroad line, and the first transcontinental train arrived in 1900 (Banks and Orlins 1981). By 1915, two railroad lines cut across the watershed.

In 1905, the City of Richmond was incorporated and the small town of San Pablo was in existence. Both communities had established schools and post offices, and many of the streets in use today had been laid out. The population size remained stable at about 23,000 people through the 1920s and 1930s, until the outbreak of World War II (Purcell 1940, cited in San Francisco Estuary Institute 2001, Banks and Orlins 1981). In the middle section of the Rheem Creek watershed, a great deal of development occurred between 1915 and the early 1940s, but the upper watershed and headwaters remained largely undeveloped, containing only a few unimproved roads.

In the early twentieth century, European, Chinese, Japanese, Mexican, Native American, and African American people migrated to the area, and the population of the watershed diversified. However, people of color had very limited economic opportunities, and most went to work for the Santa Fe Railroad company, laying tracks or cooking in the workers' camps. On the other hand, white settlers were enticed to the area with offers of business lots and unbounded job opportunities. Not surprisingly, housing was racially segregated (Banks and Orlins 1981).

Wartime Industrialization and Transformation

World War II brought major changes to the area. The Kaiser Shipyards opened in Richmond, and the employment opportunities attracted thousands of people to the area. Thousands flocked to Richmond because the shipyard work did not require any technical expertise (Moore 2000). Between 1940 and 1943, the population of Richmond more than quadrupled (Banks and Orlins 1981). Many of the people who came to Richmond to work in the shipyards were African Americans moving from the South. In 1940, there were only 270 African Americans living in Richmond; by 1943 the number had risen to 5,673, and by 1947 to 13,780 (Cole 1980). At the peak of shipyard production, 20% of laborers in the shipyards were African Americans (Moore 2000).

During the 1940s, the Rheem Creek watershed experienced rapid change and growth. By 1948, there was extensive industrial development along

Giant Highway. Between 1942 and 1948, residential development had also proceeded rapidly: new roads were built throughout the middle watershed, El Portal School was constructed in the upper watershed, roads had been laid for the Rollingwood subdivision in the upper watershed, and the headwaters became densely developed.

Throughout the region, the war marked the definitive transition of the area from agricultural to urban and suburban. Municipal infrastructure developed in Richmond and San Pablo in the late 1940s and 1950s. Streets, streetlights, and sidewalks were installed. Hospitals, shopping centers, and new schools were constructed. Fire and police departments were expanded. The remaining dairy ranchers sold their land to corporations, and farmers subdivided and sold their land for housing developments.

The 1950s

The economic opportunities of World War II ended with the war. The Shipyards closed immediately and layoffs followed quickly, reaching rates of 1,000 people per month (Moore 2000). By 1950, the overall unemployment rate in Richmond had reached 13%; for African-Americans, the rate was 29%. Although the total population of Richmond and nearby communities declined during the 1950s, the African American fraction of population actually increased; by 1960, 20% of Richmond's population was black (Moore 2000), and by 2000, 36% were black. (The percentage in the watershed is slightly higher, at 39%.)

For African Americans living in and around the Rheem Creek watershed, there was a severe shortage of decent, permanent housing. Restrictive covenants banned homeowners from selling to blacks, and temporary housing built to accommodate the growing population during the war was slated for destruction (Moore 2000). The exception to the housing crisis was Parchester Village, the sole residential community in the lower Rheem Creek watershed. In 1949, Fred Parr, a wealthy white developer, donated a large piece of property to become Parchester Village. This neighborhood of about 400 single-family homes was the first subdivision in the area where African American families could purchase their own homes. Although advertised as "a community for all Americans" (Marech 2002) and initially conceptualized as an integrated development, few whites chose to live there (Moore 2000). Instead, Parchester Village became home to many working-class and middle-class blacks, although the community has diversified in recent years (Marech 2002).

Despite the economic downturn, growth in the Rheem Creek watershed proceeded unchecked through the 1950s. Parchester Village was built in the lower watershed, as was a jetty extending out into the bay from the Breuner Marsh. In the middle watershed, Bayview School opened on the south side of the creek, and the Rancho drive-in movie theater was built just across the creek. In the upper watershed, homes were built on both sides of the creek in both the Rollingwood and College Highlands neighborhoods. Also in the upper watershed, Contra Costa College opened. Although the College offered its initial courses in a vacant Kaiser Shipyard building in Richmond, the campus moved to its permanent present day location San Pablo in 1956 (Emanuels 1986). In the headwaters, Interstate 80 had been built as a six-land highway.

The 1960s and Forward

By the end of the 1950s, the Rheem Creek watershed was almost entirely developed. Only a few changes occurred in the 1960s, including the construction of more roads in the upper watershed and the Rolling Hills Memorial Park in the headwaters. Aerial maps indicate that few changes occurred in the watershed between the late 1950s and the present day. The watershed had become – and remains today – a region of dense residential development and little remaining open space.

While the changes that took place in the Rheem Creek watershed over a two hundred year period are not unique to this watershed, they are quite remarkable in magnitude. Between the late 1700s and late 1900s, the human population of the watershed was completely transformed, from an entirely indigenous community to a diverse community of people from multiple cultures around the world. Population size and density increased dramatically. Lifestyles changed from hunting and gathering to agriculture and ranching to industry and finally to an urban lifestyle

where few people depend on the natural resource base where they live. As the human community of the Rheem Creek watershed changed, the natural environment dramatically and irreversibly altered.

Watershed Ecology

Natural Habitats

Because the Rheem Creek watershed is so heavily developed, few intact natural habitats remain today. The major exception to this pattern is the undeveloped baylands in the lower watershed. Baylands are the shallow water habitats – both lands and wetlands – that ring the San Francisco Bay and are affected by tidal action. Human activities have led to the loss of many of the historical baylands, as described on page 22.

The baylands ecosystem consists of several different habitat types; various factors influence where each habitat is found. These include the topography of the land, the movement of tides in the bay, the amount, timing, and location of freshwater inflow to the bay, and the quantity and types of sediments. Figure 15 depicts the interactions between these physical factors. In the Rheem Creek area, these factors have interacted to create broad baylands with moderate salinity and a relatively large amount of suspended sediment in the water. The remaining baylands in the lower Rheem Creek watershed consist of tidal flat and tidal marsh along the shoreline (Goals Project 1999). Both tidal marsh and tidal flat are important habitats of the bayland ecosystem.

Tidal flat habitat is the area between the mean lower water level of low tides (called MLLW) and the mean tide level (MTL). The tidal flat is covered twice a day by high tides. This habitat type was historically found along the seaward edge of tidal marshes. Mudflats, sandflats, and shellflats are all types of tidal flats, although mudflats are most common in the Bay Area. Mudflats support plant and animal species such as shellfish, green and red algae, worms, diatoms (microscopic or unicellular algae), eelgrass, and sea lettuce. During high tide, tidal flats are covered by Bay water, providing habitat for many fish to forage. At low tide, tidal flats are important feeding areas for shorebirds such as the semipalmated plover, American avocet, willet, marbled godwit, western sandpiper, and dunlin (Goals Project 1999). A long tidal flat remains today in the lower Rheem Creek watershed.

Figure 15: Factors Influencing Distribution of Baylands Habitats

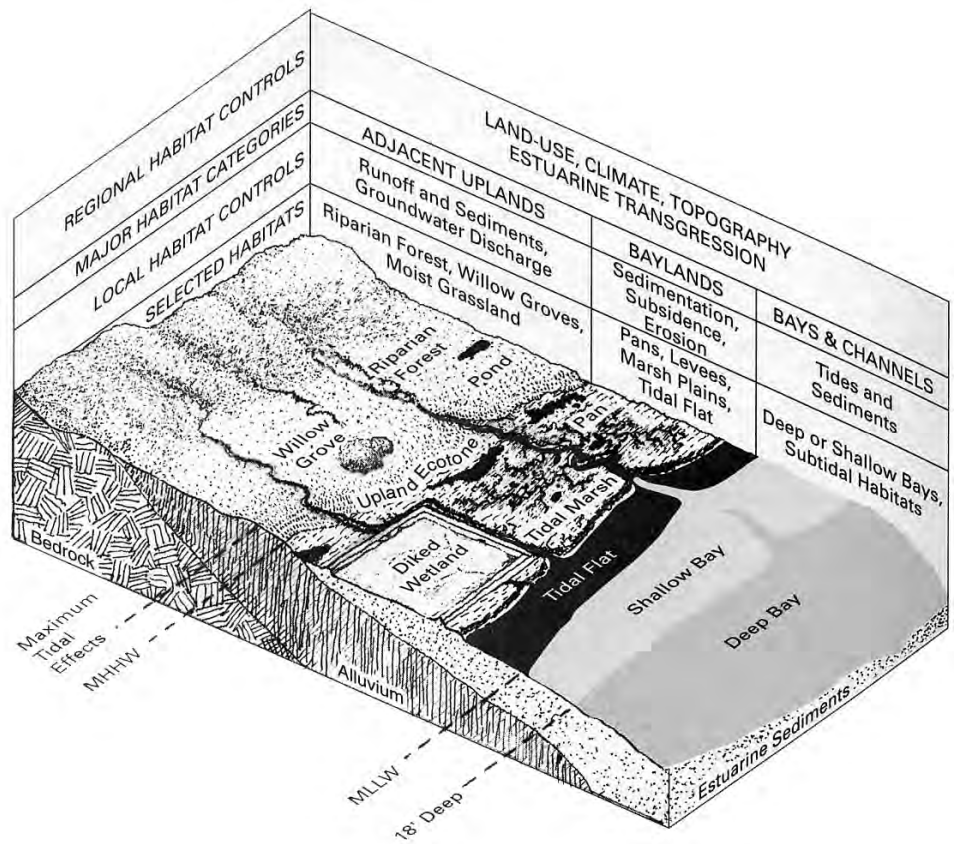


Figure Source: Goals Project 1999

Tidal marsh is a vegetated wetland that receives some tidal action, but unlike tidal flats it is not completely inundated at high tide. Tidal marshes typically have three zones of vegetation which are influenced by the height of the tides and the distance from shore. Characteristic tidal marsh

The Ecological Importance and Historical Loss of the Baylands Habitats

The baylands are the shallow water habitats – both lands and wetlands – around the San Francisco Bay. These are the lands that are in contact with tides or that would be in contract with tides had levees and other structures not been built. They sit between the many watersheds that drain into the San Francisco Bay and the deep waters of the open bay.

The baylands have great ecological importance. They provide habitat for many species, such as salmon, seals, egrets, ducks, salt marsh harvest mice, and the California clapper rail. Wetlands within the baylands provide additional benefits. Wetlands are areas that are covered by shallow water or where the water table is at or near the surface. By filtering pollutants from the freshwater flowing into the bay, wetlands help to improve water quality, protect upstream lands from floods, and protect shorelines from erosion.

The baylands have disappeared as the human population of the San Francisco Bay Area has increased and as land use practices have changed. With the arrival of Europeans in the San Francisco Bay Area, the baylands were used for grazing cattle and sheep. In the mid-1800s, large sections of the bay were filled to provide land for railroad lines, roads, and ports. Around the same time, farmers began to dike and drain the baylands and convert them to agricultural uses. These human alterations to the natural landscape have had the following effects:

Tidal marsh habitat – the vegetated wetlands subject to tidal action – has decreased from 190,000 acres to 40,000 acres due to the filling and diking of the bay. 79% of pre-1850s tidal marsh has been lost.

Tidal flat habitat – the mudflats, sandflats, and shellflats that are inundated at high tide – has decreased from 50,000 acres to 30,000 acres due to fill, erosion, reclamation, and conversion of tidal flat to tidal marsh. This is a, 40% loss.

Moist grassland/vernal pool habitat – an area of annual grasslands with shallow surface depressions that hold water during the rainy season and dry during the dry season – has decreased from 24,000 acres to 15,000 acres due to farming and urban development., a loss of 37%.

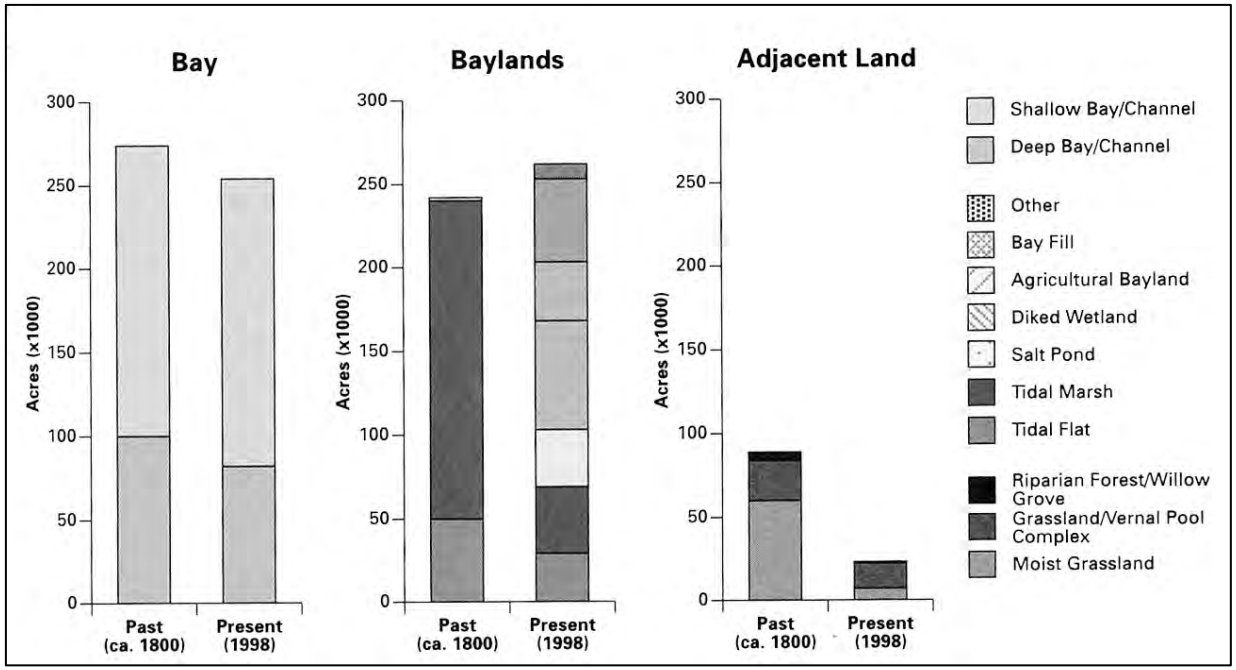


Figure Source: Goals Project 1999

As a result of the loss of baylands habitats, 51 species that live in or near the baylands are listed as threatened or endangered. The Rheem Creek watershed contains suitable habitat for at least three of these species: the salt marsh harvest mouse, California clapper rail, and California black rail.

Source: Goals Project 1999.

plants are Pacific cordgass and pickleweed, which are adapated to tolerate the tidal saltwater and grow in saline soil. Healthy tidal marshes provide habitat for many species: fish such as gobies, sculpins, and three-spined stickleback, birds such as snowy egret, northern harrier, California clapper rail, California black rail, willet, short-eared owl, salt marsh yellowthroat, and San Pablo song sparrow, and small mammals like the salt marsh harvest mouse and salt marsh wandering shrew (Goals Project 1999). While much of the tidal marsh in the Rheem Creek watershed has been filled and replaced by non-native grasslands, Giant Marsh to the north of the channel is an intact tidal salt marsh. Non-native annual grassland is another habitat type found in the lower Rheem Creek watershed, upstream of the tidal marsh. Not strictly a bayland habitat type, grassland is considered an associated upland habitat. The dominant plant species in the grassland are Italian ryegrass, soft chess, wild oats, and ripgut brome. Although this grassland consists of invasive species that have crowded out the native species, it is still important habitat for many native animal species, including the tiger salamander, gopher snake, a variety of migratory shorebirds, turkey vulture, red-tailed hawk, burrowing owl, American kestrel, northern harrier hawk, black-shouldered kite, ornate shrew, California ground squirrel, western harvest mouse, and California vole.

Small patches of seasonal wetlands remain interspersed throughout the non-native grassland in the lower watershed (Impact Sciences, Inc. 2002). These wetlands are poorly drained depressions that fill with fresh water during the rainy season. Some of these wetlands are former tidal marshes that were closed off from tidal action due to fill. Typical plants are bulrush and cattails. Wildlife that inhabit seasonal wetlands include ducks, shorebirds, egrets, and herons.

In the heavily urbanized middle and upper sections of the watershed, no substantial segments of natural habitat remain.

Non-Native Species

Non-native species have replaced native species throughout the watershed. Non-native plants often spread quickly, crowd out native vegetation, and often provide less habitat value than the vegetation replaced. They can often be very difficult to control due to long-living seeds, rapid growth rates, and other adaptations that allow them to establish and out-compete native plants. The lower watershed consists almost entirely of non-native grassland. The non-native species in the middle and upper watershed include eucalyptus trees, blackberry, and ivy (Walkling 2004).

Introduced terrestrial animals are also found in the watershed. These include the Norway rat, the red fox, and feral cats. These animals prey upon native birds and small mammals living along the bay. The presence of nonnative predators poses an especially serious threat to native waterfowl (San Pablo Bay Watershed Restoration Program 2000).

Endangered and Threatened Species

With its intact tidal flat and tidal marsh, the lower Rheem Creek watershed contains suitable habitat for at least three of the 51 baylands species that are listed as threatened or endangered under the state or federal Endangered Species Acts: the salt marsh harvest mouse, the California clapper rail, and the California black rail. All three species are known to occur in Giant Marsh. (See page 24.) Two additional species that have been documented in the lower watershed, the San Pablo vole and salt marsh wandering shrew, are listed as species of special concern (Martinelli 2004). These are animals that are not listed under the federal or state Endangered Species Act but which are declining at a rate that could result in their being listed or which historically occurred in low numbers and known threats to their persistence currently exist. Suitable habitat also exists for a number of other species of concern, including the osprey, western burrowing owl, short-eared owl, and northern harrier (Impact Sciences, Inc. 2002).

All of these species of concern depend on the tidal marsh habitat for their survival. In the Bay Area, only about 20% of historical tidal marshes remain. Marshes have been filled to allow development; converted to brackish (less salty) marshes because of freshwater discharged from sewage treatment plants; invaded by non-native species such as

Endangered and Threatened Species of the Rheem Creek Watershed

Salt Marsh Harvest Mouse

The salt marsh harvest mouse is a tiny mouse – with a combined head and body length of just three inches and an average weight of less than half an ounce – native to the salt marshes of San Francisco Bay, including the tidal salt marsh of the lower Rheem Creek watershed. The harvest mouse is listed as an endangered species by both the state and federal governments. The extensive loss of tidal marshes throughout the Bay Area and predation by non-native species such as red fox and feral cat are the major causes of the mouse's endangerment.

Photo Source: US EPA Paul Kelly

The salt marsh mouse is a "cover dependent species," meaning that it lives under thick vegetation. The harvest mouse depends upon the thick perennial plants of the salt marsh for protection and moves inland to grasslands to escape the highest winter tides or when the grasslands provide greater cover than salt marsh vegetation (during the spring and summer). In particular, the mouse prefers the middle and upper portions of the tidal marsh, where pickleweed and other salt-tolerant plants grow. For food, the mouse depends on pickleweed and saltgrass for most of the year, though in winter it consumes fresh grasses.

With the extensive loss of tidal marshes, there is little remaining natural habitat for the harvest mouse in the Bay Area. Even those tidal marshes that remain are often small narrow strips without upper and middle zones that provide cover for the harrvest mouse to escape high tide or to hide from predators.

California Clapper Rail

The California clapper rail, a secretive hen-like waterbird, is an endangered species found in the lower Rheem Creek watershed. The clapper rail is 13-19 inches from bill to tail. The rail relies on the tidal marsh for cover and the tidal flat for food. At low tide, the rail forages in the low marsh and tidal flat area, consuming invertebrates such as mussels, clams, snails, worms, spiders, insects, and fish. At high tide, the rail depends on the higher marsh area, where there is dense vegetation such as pickleweed, saltgrass, and cordgrass, for nesting and refuge.

The rail population has declined due to a number of human-related influences. The most immediate threat is predation. At least thirteen native and nonnative species of mammals and birds prey on the bird or its eggs. Nonnative predators include the red fox, feral cats, and the Norway rat. The predation problem is exacerbated by development, which has reduced the population size of predators that would normally prey upon the animals that eat the clapper rail, such as the red fox. Electric transmission lines that cross tidal marshes provide new perches for native raptors that prey upon the rail. The clapper rail is also threatened by the loss of 85% of the tidal marshes in the San Francisco Bay. The small size and fragmentation of remaining marshes further limits the availability of suitable habitat. Commercial and sport hunting during the late 1800s have also contributed to the decrease in the rail population. Finally, the rail is threatened by the accumulation of heavy metals, notably mercury, in bird eggs.

Photo Source: USFWS Mike Boylen

California Black Rail

The California black rail is a tiny, sparrow-sized bird that lives in saltwater, brackish, and freshwater marshes. Historically found from Central California south to Baja, the black rail population is now limited to the tidal marshes of the northern San Francisco Estuary, including the lower Rheem Creek watershed. The rail is listed as a threatened species under California's Endangered Species Act and as a "species of special concern" under the federal Endangered Species Act.

Photo Source: USFWS Rich Stallcup

The black rail relies on tidally influenced, fully vegetated, upper salt marsh habitat dominated by pickleweed. The rail nests below the pickleweed, preferring a dense canopy cover for protection. The rail prefers habitat where tidal action is unrestricted, that is fully tidal and undiked marshes. In addition, the rail requires adequate upland refuge for protection during high tides. Aquatic invertebrates and terrestrial insects from the ground are the rail's primary food sources.

The black rail is threatened by the widespread reduction and elimination of the transition zone between the marsh and uplands, on which the rail depends. The rail's habitat has been fragmented, degraded, and lost due to flood-control projects, urbanization, agriculture, and livestock grazing. Predation by rats, feral cats, and red foxes also threaten the black rail.

Sources: Goals Project 2000, United States Fish and Wildlife Service 2004, California Department of Fish and Game 2000, Shellhammer 1998, Live Oak Associates 2002

cordgrass; and polluted by urban run-off, industrial discharge, and sewage effluent. With the loss of tidal marshes throughout the Bay Area, few places remain where animals dependent on this habitat type can find food or cover.

In addition to habitat loss, the fragmentation of remaining tidal marsh habitat has also contributed to the decline in these species (and others) throughout the Bay Area. The process and impacts of habitat fragmentation are described in the text box "Habitat Fragmentation and Species Survival" (right).

Ecological Changes in the Watershed

The earliest accounts of western Contra Costa County described an area rich in fish, waterfowl, deer, elk, and other wildlife. The Huichun depended on these resources, and it is likely that their impact on these abundant resources was relatively minimal, due to their small population and their hunting and gathering lifestyle. The San Francisco Estuary Institute's EcoAtlas Baylands Maps (1998) present the earliest and most thorough picture of the natural Bay Area landscape, based on hundreds of data sources from the eighteenth and nineteenth century, including maps, sketches, paintings, oral histories, journals, and interviews. Although there is considerable uncertainty about pre-European habitat conditions at the local scale, the historical ecological description of the Rheem Creek watershed presented here is based on these maps.

Prior to the arrival of the Spanish in the Bay Area, a much broader tidal flat and tidal marsh spanned the shoreline in the lower Rheem Creek watershed. The tidal flat and tidal marsh supported large populations of waterfowl, shorebirds, gulls, rails, egrets, herons, raptors, turtles, frogs, salamanders, snakes, seals, and rodents. Agriculture, grazing, filling of the bay, and urban development all contributed to the massive loss of tidal flat and tidal marsh habitat. While tidal flat and tidal marsh habitats in the watershed have been substantially reduced, several habitat types that once existed – such as native grasslands, vernal pools, and riparian habitats – have nearly ceased to exist.

Habitat Fragmentation and Species Survival

Habitat fragmentation is the "process whereby a large, continuous area of habitat is both reduced in area and divided into two or more fragments" that remain isolated from each other. Frequently, these fragments are surrounded by a landscape that has been heavily altered by human activities, such that the fragments are like "habitat islands in an inhospitable human-dominated sea." Habitat fragmentation threatens the survival of species in several ways.

First, habitat fragmentation increases the amount of "edge habitat" (the habitat that is adjacent to human activities) relative to the total habitat of the fragment. This increase leads to a number of "edge effects." For example, with a greater proportion of edge habitat, the fragments become more vulnerable to invasion by non-native species and domestic animals that can prey on native species within each fragment. In addition, an increase in edge habitat leads to greater variability in the microenvironment – conditions such as temperature, light, humidity, and wind – at the edge of a habitat fragment; species adapted to tolerate very specific environmental conditions may be forced into the interior of the fragments in order to survive. Because there is less interior habitat in fragmented landscapes, there is simply less area in which these especially adapted species can live.

Another problem associated with habitat fragmentation is that it may reduce an animal's ability to forage as broadly as necessary. Species that move around the landscape to feed on widely dispersed or seasonal food sources may no longer be able to find the resources on which they depend.

Habitat fragmentation may also lead to the decline of populations because it isolates a species into subpopulations with restricted areas. These reduced populations then become more vulnerable to a number of problems associated with small population size, including inbreeding.

In the San Francisco Bay Area, many of the remaining baylands habitats, including the tidal marsh, are now fragmented due to development, fill, and levees around the margin of the bay. High quality tidal marsh habitat includes wide buffers that minimize human disturbance and predation by non-native species, as well as connections to other large patches of marsh so that birds and mammals can move safely between them. The protection and restoration of the undeveloped former tidal marsh in the lower Rheem Creek watershed can help to alleviate the detrimental effects of habitat fragmentation, thereby better protecting the threatened and endangered species dependent on this habitat

Sources: Primack 2000, Goals Project 1999.

Before grazing and agriculture came to dominate the landscape in the 1800s, the lower and middle Rheem Creek watershed contained an extensive moist grassland, a habitat dominated by perennial grasses and sedges. In the Rheem Creek watershed and throughout the Bay Area, moist grasslands were found on valley floors and on hills facing southwest. Perennial bunchgrasses, Italian rye grass, Baltic rush, iris-leaved rush, Santa Barbara sedge, and creeping wild rye were the dominant plant species. Animal species found in this habitat included western toad, western skink, meadowlark, horned lark, savannah sparrow, and western harvest mouse (Goals Project 1999, Estuary Institute 1998).

In the lower watershed, vernal pools were found interspersed throughout the grassland. Vernal pools are shallow surface depressions that fill with and hold rainwater and nearby runoff during the rainy season and dry out by evapotranspiration during the dry season. Vernal pool species include fairy shrimp, tadpole shrimp, California tiger salamander, western spadefoot toad, common garter snake, black-necked stilt, American avocet, California vole, and black-tailed hare. Most of the vernal pools in the Bay Area have been degraded by farming and filling (Goals Project 1999). Today only remnants of vernal pools remain, scattered throughout the lower watershed.

Natural riparian habitat once bordered the edge of Rheem Creek. The habitat served important ecological functions, including regulating the temperature of water in the stream, protecting water quality, controlling erosion, moderating downstream floods, and providing food and refuge for wildlife (San Pablo Bay Watershed Restoration Program 2000). The natural riparian habitat is likely to have included cottonwood, sycamore, willow, buckeye, coast live oak, and California bay trees. Animal species such as the Pacific treefrog, California newt, striped skunk, raccoon, coyote, black-tailed deer, wood duck, downy woodpecker, great-horned owl, tree swallow, scrub jay, and song sparrow all depended upon the riparian corridor (Goals Project 1999). The upper watershed likely supported an oak woodland plant community dominated by coast live oaks and including madrone and bay trees.

With the creation of the Rancho San Pablo, the natural habitats in the watershed were substantially altered. The introduction of agriculture and grazing throughout the watershed led to the loss of moist grasslands and may have sparked the spread of non-native plant species. As the watershed urbanized in the twentieth century, oak woodlands, riparian habitat, and the grasslands were paved over and converted to residential and industrial lots. Non-native species, introduced for landscaping, have spread throughout the watershed. Even the remaining natural habitats are now largely isolated from one another and fragmented, as discussed earlier.

Watershed Contamination

Over the past century, the middle and lower watershed have housed various manufacturing facilities. According to the United States Environmental Protection Agency (EPA), about ten hazardous waste facilities exist within the watershed. These are specific facilities regulated by the EPA that handle materials that can pose a hazard to human health or the environment if improperly managed.

Although hazardous waste facilities are now tightly controlled, the environmental byproducts of manufacturing were largely unregulated prior to the 1960s. As a result, many soils and water supplies throughout the country became polluted. One example in the Rheem Creek watershed is the former site of American Standard Products on the boundary of the middle and lower sections of the watershed. From 1913 to 1967, American Standard Products, now the world's largest manufacturer of bath and kitchen products, operated a porcelain manufacturing facility on Giant Highway, next to Rheem Creek. The site included the factory itself as well as a waste disposal area on the bank of Rheem Creek. The soil and groundwater were found to be contaminated with lead, zinc, and petroleum byproducts. By the mid-1990s, American Standard had undertaken multiple actions to clean up the site. These have included removing soils and underground storage tanks, removing trees and brush from the site, placing a cap on remaining contaminated soils, and lining the creek with reinforced concrete, (California Department of Toxic Substances Control 2004). These measures are believed to have

remediated the contamination problems, although regular testing of groundwater continues at the site (Sunga 2004). Other contaminated sites in the lower watershed have also been identified and remediated. Unfortunately, little systematic data about current groundwater and soil contamination on a watershed scale is available.

We have seen in this chapter that the Rheem Creek watershed has undergone a complete transformation in the past 250 years. Today, the Rheem Creek watershed is a heavily developed area with only a few vestiges of its ecologically diverse past. Ecological diversity has given way to human diversity as the watershed has urbanized and its population has expanded and adapted to changing social and economic conditions. Not surprisingly, these major changes throughout the watershed have had profound impacts on Rheem Creek itself. The stream channel is the focus of the next chapter.

III. THE STREAM CHANNEL

In many ways, Rheem Creek is typical of urban streams throughout the world. Development and urbanization throughout the watershed have greatly altered the amount, timing, and quality of water in the channel as well as the animals that once inhabited the creek. Development pressures have also led to major changes in the physical structure of the creek channel itself, which now mostly runs in a concrete channel. This chapter describes the channel today – its physical structure, hydrology, water quality, and biotic community – as well as the changes that have led to its current conditions.

Structure and Conditions of the Channel

Rheem Creek runs 3.4 miles from its headwaters to its mouth at the San Pablo Bay. It has just one short tributary that flows in from the north and meets the main channel at the lower end of the Contra Costa College Campus. Historical maps indicate that there was once another tributary in the lower watershed that was filled for development in the 1950s (Urban Creeks Council of California and FarWest Restoration Engineering 2002).

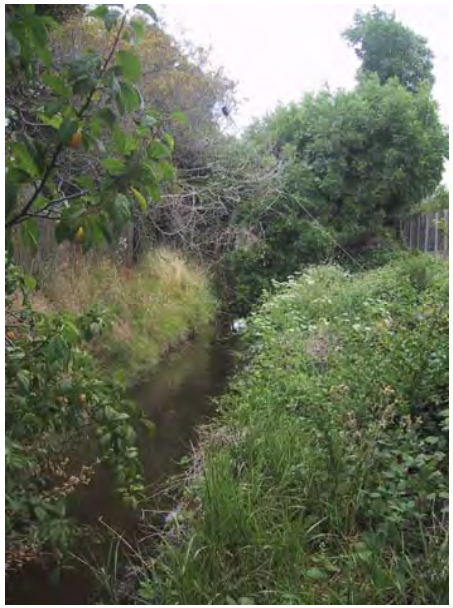
Rheem Creek is a highly modified channel. Today, the creek runs in underground culverts for 32% of its total length and above ground for 68% of its length. Although many creeks have been placed in underground culverts throughout the Bay Area, in Contra Costa County, creeks run underground for an average of 8% of their length (Contra Costa County Community Development Department and the Contra Costa Watershed Forum 2003). Therefore, Rheem Creek differs from the "average" Contra Costa County creek in having four times as much of its length in underground culverts.

Throughout this century, the sections of Rheem Creek that remain above ground have been altered for various flood control projects. Of the length of the creek above ground, 18% is contained within a concrete channel, 46.5% is within a constructed earth channel, 34% is within a "natural" channel, and 1.5% is in a channel with riprap (a layer of large stones and broken rock or concrete placed in a streambed or on the sides of a channel to protect it from erosion). It is important to note that even the so-called "natural" channel is heavily impacted by human development. In the "natural" reach, the creek is overgrown with non-native plants and narrowly confined between rows of houses. A segment-by-segment description of the channel follows.

Headwaters: From its headwaters in El Sobrante, the creek runs as a natural channel for a brief stretch. Just before passing under Interstate 80, the channel runs into an underground culvert.

Upper watershed: West of I-80, the creek emerges from a concrete box culvert and flows in an unpaved confined channel. It is contained in a narrow right-of-way behind homes, visible to the public only where it passes under Fordham Street (Figure 17). Here Rheem Creek forms the political boundary between the City of Richmond (the College Highlands subdivision) and unincorporated Contra Costa County land (the Rollingwood neighborhood). Although some native cottonwood trees still exist alongside the channel, for the most part the channel is overgrown with non-native species of plants.

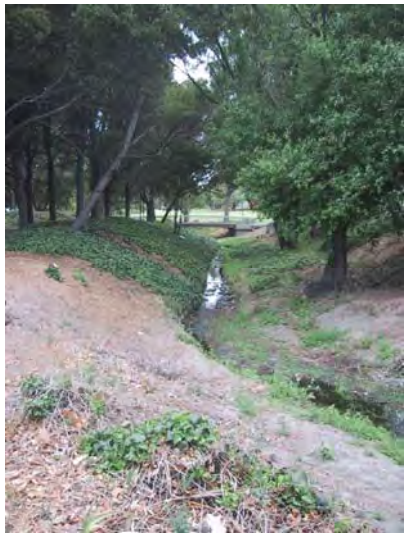
Figure 16: Rheem Creek in the Upper Watershed



Flowing out of this residential area and onto the Contra Costa College campus, the channel widens significantly. On the Contra Costa College campus, the creek is in a natural, unconfined channel and is easily accessible to the public (Figure 18). Non-native plants such as ivy and eucalyptus cover the banks and shade the channel. The single natural tributary of the creek flows from the north and joins the creek on campus, but much of the tributary runs

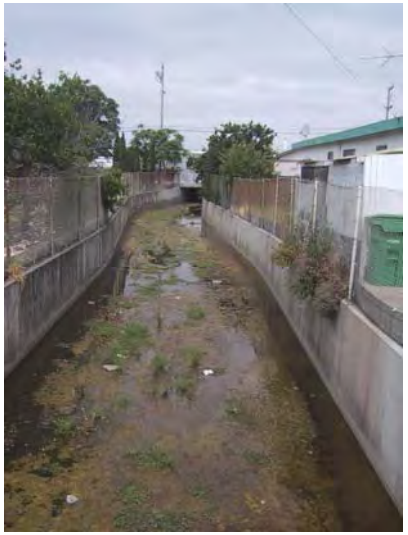
underground. Below Contra Costa College, the creek passes into an underground culvert that crosses beneath San Pablo Avenue.

Figure 17: Rheem Creek on the Contra Costa College Campus



Middle watershed: In the middle section of the watershed, the channel is alternately contained in a wide, smooth engineered concrete channel and a constructed earthen channel. Although the creek is visible, fences throughout the middle watershed prevent access to the channel (Figure 18) West of San Pablo Avenue, the creek emerges from its underground culvert and flows in a concrete box channel through the future site of Wanlass Park and into a residential development in San Pablo. It passes underneath bridges on residential streets and through the yard of the Bayview Elementary School.

Figure 18: Rheem Creek in the Middle Watershed



Lower watershed: As it passes into and through the lower watershed, the undeveloped baylands lands west of Giant Highway, Rheem Creek runs in a trapezoidal earthen channel with levees alongside both banks Here the creek has been straightened and narrowed (Urban Creeks Council and FarWest Engineering 2002). Most of the creek bed in this lower reach is lined with riprap along its banks. Nonetheless, riparian vegetation – including some native species – line the creek (Figure 19).

Historical Changes to the Channel Structure

Rheem Creek has been modified over most of its length due to urbanization and development throughout the watershed. Historical maps from the United States Geological Survey (USGS) indicate that Rheem Creek formerly contained three tributaries flowing in from the north; only one of these tributaries remains today, and it has been shortened from its original extent.

Figure 19: Rheem Creek Flowing to the Bay

In addition, the land area now drained by Rheem Creek was drained three separate channels flowing into the San Pablo Bay until the early 1950s. Intensive urbanization of the watershed in the 1950s, including the construction of Parchester Village, led to the loss of these historical channels. Most likely, the creeks were paved over and replaced with underground storm drains to convey water.

In many places, including the Rheem Creek watershed, urbanization proceeded with little consideration of potential flood risk. Homes and roads have been placed close to streams, leading to landowner demands for flood control actions. Today, legislation requires developers to consider the environmental impacts of new construction, and many cities have ordinances requiring new homes to be built a certain distance from the creek. While many new developments incorporate creeks as visual amenities, several decades ago creeks were treated more as a potential threat to be contained. Channel modifications were undertaken with the single purpose of moving water quickly to avoid flooding problems on neighboring property.

In response to landowner demands and flooding problems, many city and county governments have undertaken flood control measures such as the removal of native riparian vegetation, building a larger channel that will carry more water, and creating a smoother channel surface – such as rock or concrete – to move water more quickly. These flood control "improvements" have eliminated nearly all riparian vegetation that once flourished along the margins of Rheem Creek. Although city and county agencies with jurisdiction in the watershed do not have records about all flood control projects on Rheem Creek, existing records reveal that most of these flood control projects were designed and constructed between the mid 1950s and early 1960s.

In the upper watershed, the channel was altered in the mid-1950s, when the College Highlands subdivision was built in the City of Richmond. The channel appears to have been narrowed, most likely to allow for larger lots, and made smoother to move water more quickly through the neighborhood.

In the lower watershed, a large flood control project in 1960 substantially altered the channel. This project was undertaken by the United States Army Corps of Engineers (Corps) in response to a 1955 flood that had caused over $20,000 worth of damage to homes and businesses in the City of San Pablo. The Corps conducted a benefit-cost analysis before initiating the $413,000 project and found that the benefits outweighed the costs by a ratio of 1.8 to 1. A major reason for carrying out the project was to facilitate the development of "higher end" homes on 15 undeveloped lots along the creek and on 200 acres of undeveloped industrial property (United States Army Corps of Engineers 1958). This project consisted of placing 1.5 miles of Rheem Creek in a constructed channel in the lower watershed. Eighty percent of the project consisted of a trapezoidal, unlined earth channel, while the remaining twenty percent - in the more congested areas with higher real estate costs - was a concrete-lined rectangular channel. The project also constructed new facilities to carry drainage from adjacent areas into the channel. The open ditches, drains, and pipe culverts were built to protect the community from future flood damage (United States Army Corps of Engineers 1962). In addition, the entire lower section of the creek was relocated about 300 feet to the south (United States Army Corps of Engineers 1960).

The Corps determined that the "only feasible method of providing flood protection along Rheem Creek is by increasing the channel capacity" (United States Army Corps of Engineers 1958). The engineered channel was designed to accommodate a flow of 800 cubic feet per second (cfs) at Giant Highway, which was believed to be 1.5 times more flow than had caused the 1955 flood (United States Army Corps of Engineers 1962). The "design discharge" of 800 cfs is about two times the highest flow recorded in the creek during the 30 years (1960-1990) that a gage recorded streamflow..

To give a visual sense of changes to the creek channel, pages 31 and 32 offer a photographic comparison of the Rheem Creek watershed with a nearby, less developed watershed.

Streamflow

All of the streamflow records for Rheem Creek come from a United States Geologic Survey (USGS) stream gauge on lower Rheem Creek, just west of Giant Highway (see Figure 2). The gauge operated from 1960 to 1990. The gauge on Rheem Creek measured runoff in the upper 1.49 square miles (53%) of the watershed and did not account for runoff entering the system from the lands west of Giant Highway.

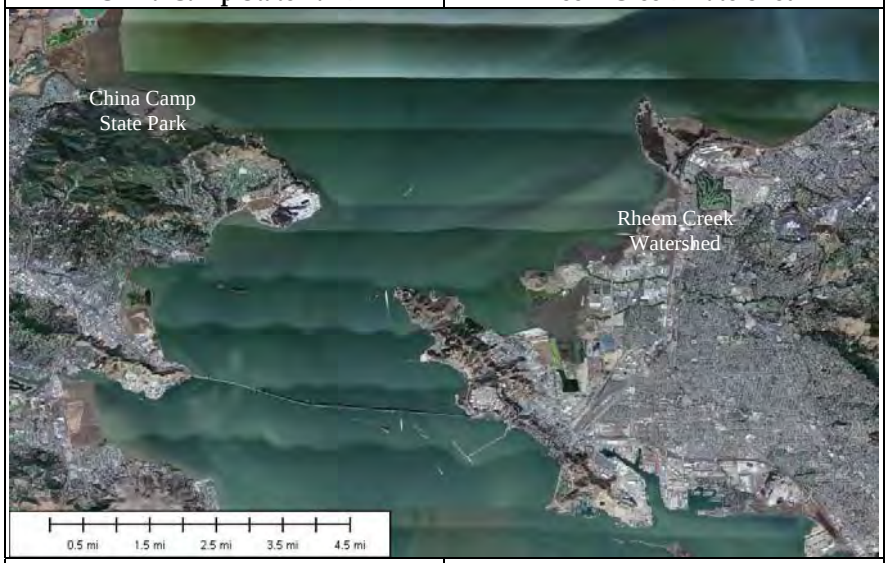
Because little additional development has occurred in the watershed and no major changes have been made to the channel since the USGS discontinued this gauge, it is likely that the streamflow recorded by the gauge is similar to the streamflow in the channel today. It is important to note that this flow record is limited, and it is difficult to have a complete picture of the flow patterns in the creek with only thirty years of gauge records.

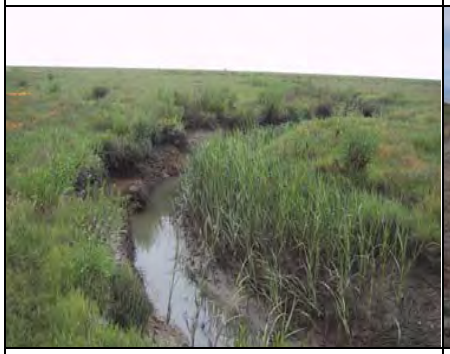
Typical Streamflow Patterns

The annual hydrograph is a record of streamflow over a one-year period and illustrates how daily streamflow changes throughout the seasons of a

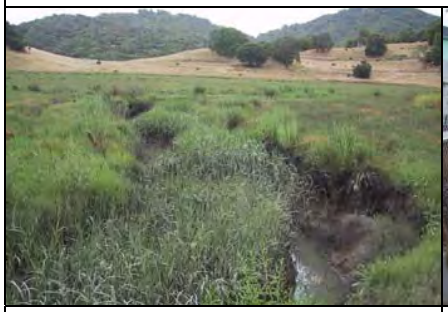
Changes to the Rheem Creek Channel and Watershed

The paired photos below show China Camp State Park on the left and Rheem Creek on the right. China Camp State Parks is located directly west of Rheem Creek watershed on the opposite shoreline of the San Pablo Bay. Though not perfectly analogous, the small watersheds in China Camp State Park are perhaps the best reference watersheds in the area. While by no means scientific, this photographic comparison can provide a sense of what the Rheem Creek watershed may have looked like before it was developed.

China Camp State Park	Rheem Creek Watershed
	
Length of Creek: 1 mile	3.4 miles
Area of watershed: 0.7 square miles	2.8 square miles
Highest elevation: 300 ft	250 feet
Average Annual Rainfall: 35 inches	22.5 inches
Population: 0	12,700
Landuse: Open space (100%)	Residential homes (40%), Public or semi-public uses (22%), Parks and open space (22%), Industrial (7%), Other (7%)

China Camp State Park	Rheem Creek Watershed
Lower Watershed	
Above: Marsh channel in China Camp	Above: Rheem Creek as it enters San Pablo Bay
Above: Marsh as seen from Turtle Back Hill	Above: Giant Marsh as seen from Pt. Pinole
	
Above: Marsh channel entering San Pablo Bay	Above: Rheem Creek entering San Pablo Bay

Changes to the Rheem Creek Channel and Watershed (continued)

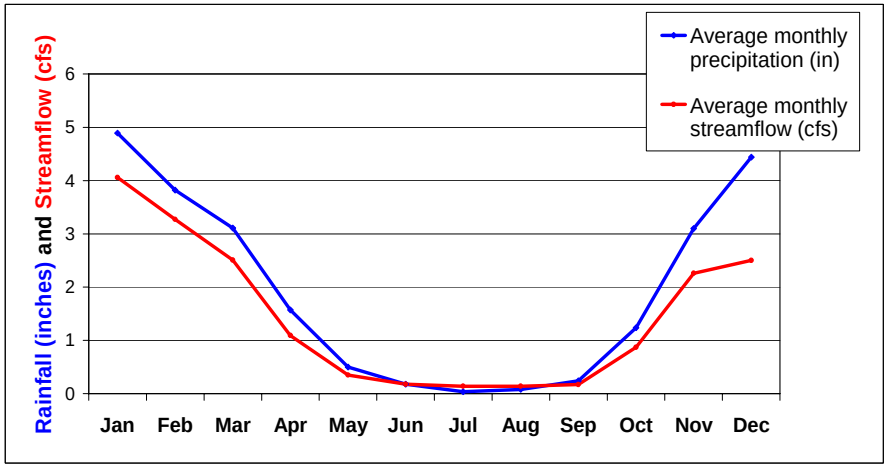
China Camp State Park	Rheem Creek Watershed
Middle Watershed	
	
Above: Creek in China Camp	Above: Rheem Creek in San Pablo
Upper Watershed	
Above: Creek in China Camp	Above: Rheem Creek at Contra Costa College
Above: Creek in China Camp	Above: Rheem Creek in Rollingwood

China Camp State Park	Rheem Creek Watershed
Headwaters	
Above: Bay laurel forest at top of China Camp	Above: Rollingwood Memorial Cemetery

given year. Figure 21 shows that the average annual pattern of streamflow in the creek is quite similar to the average annual rainfall pattern. Flow in the creek is greatest during the winter months of January and February. In very dry years the creek bed may have no flow at all during the summer months, and even in wet years, there is typically little water in the creek between May and September.

Figure 20: Average Monthly Rainfall in Richmond, CA (1950-2003) and Average Monthly Streamflow in Rheem Creek (1960-1990)

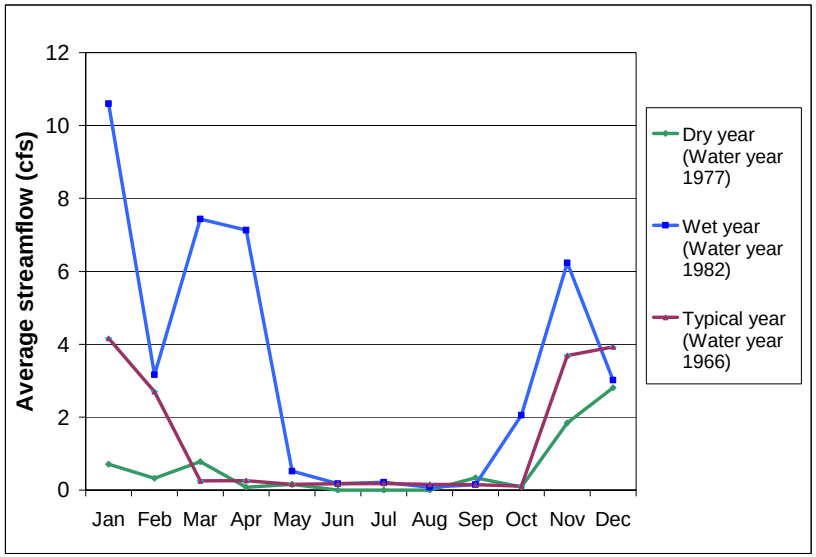
It is interesting to note that while the most rain falls in December and January, the creek reaches its highest level in January and February. This



suggests that early in the rainy season, rainfall infiltrates into pervious ground surfaces. As the rainy season continues and soils in the watershed become saturated, rainfall begins to run off the land and into the creek.

Wide variation in annual rainfall in the watershed results in large variability in the amount of water flowing through Rheem Creek during a given year, as illustrated in . The average daily streamflow each month in Rheem Creek for a wet year, a dry year, and a year with average rainfall were very different. In January of 1982, the wettest year on record, the creek had an average daily flow of 10.6 cubic feet per second (cfs), while in January of 1977, a much dryer year, average daily flow in the creek in January was only 0.6 cfs.

Figure 21: Average Streamflow by Month in Rheem Creek (Dry, Wet, and Typical Years)



Peak Flow

Peak flow is the highest flow recorded in a creek each year. Peak flows in Rheem Creek have varied widely in magnitude, as shown in Figure 23. While average flow in the creek is about 290 cfs, a peak flow as high as 477 cfs occurred in 1969, and the peak flow was only 99 cfs in 1976. Peak flow occurs most frequently during the month of January. Many peak flows have also occurred during the months of November, December, and February. Figure 24 shows the timing of peak flow in the creek.

Rheem Creek Streamflow in Perspective

Compared to nearby creeks in Contra Costa County, Rheem Creek is quite small. Figure 24 and 26 compare the mean annual streamflow of Rheem Creek with the mean streamflow of two nearby creeks, Wildcat Creek to the south and Pinole Creek to the north.

Figure 22: Peak Flow in Rheem Creek (1961-1990)

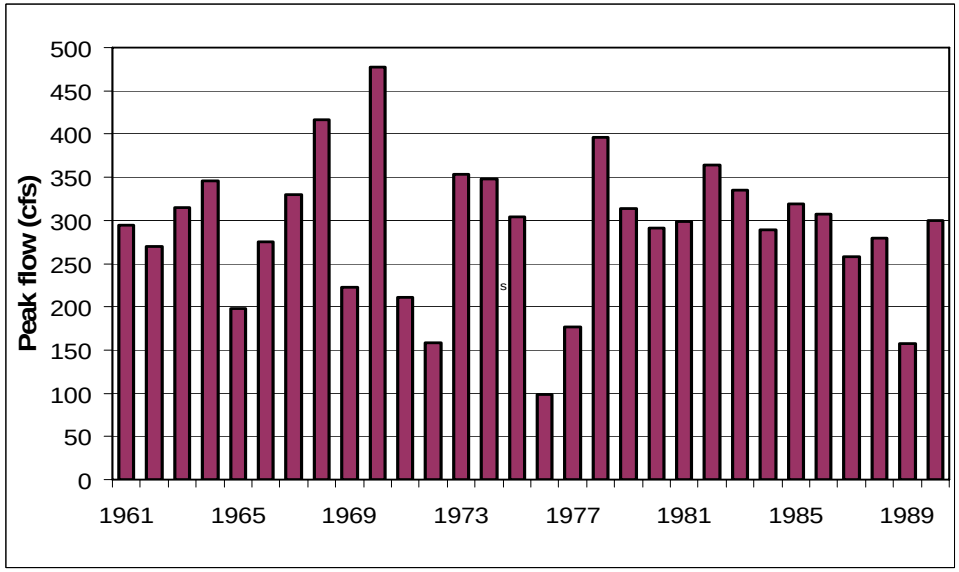


Figure 23: Timing of Peak Flow in Rheem Creek (1961-1990)

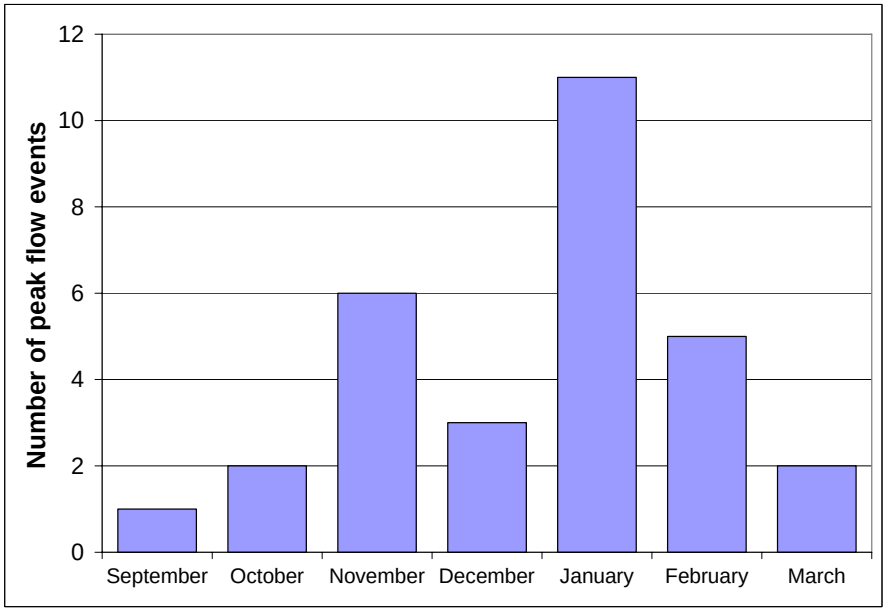


Figure 24: Mean Annual Streamflow in Rheem, Wildcat, and Pinole Creeks (1965-1974)

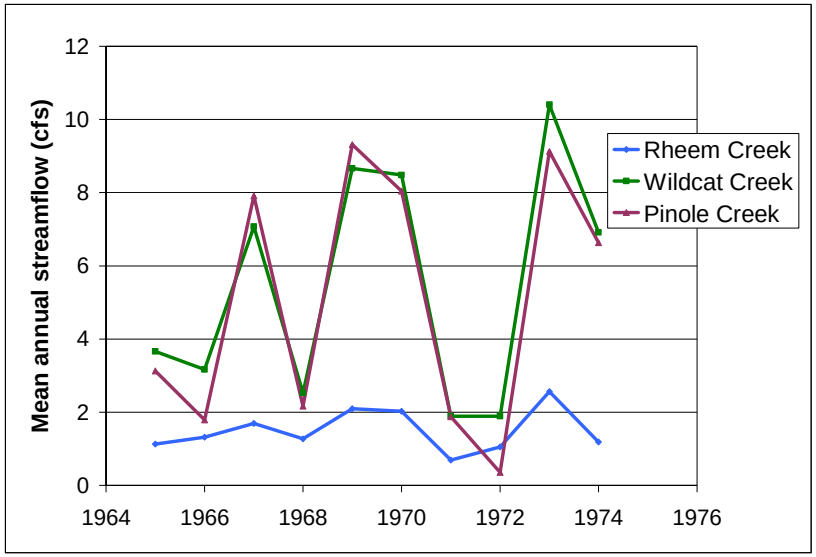


Figure 26: Mean Annual Streamflow per Unit Drainage Area in Rheem, Wildcat, and Pinole Creeks (1965-1974)

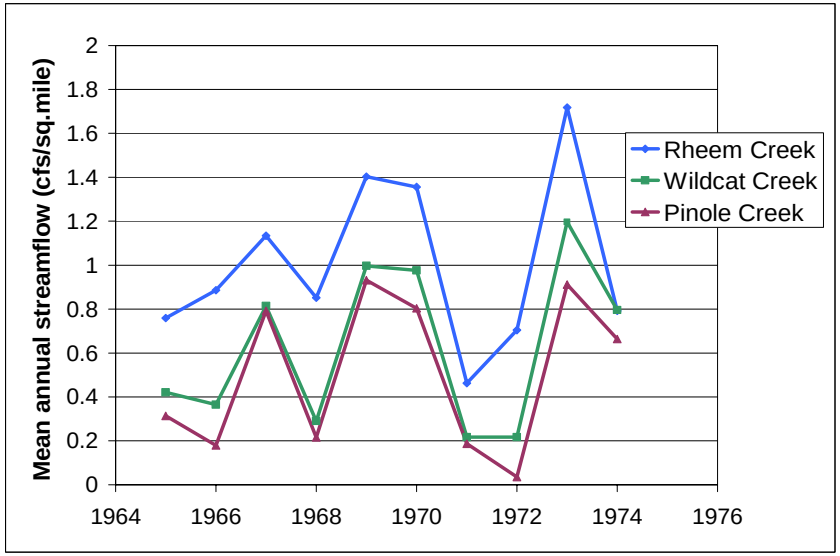


Figure 25 shows that the two neighboring creeks contained three to four times more flow than Rheem Creek during the ten years when the operation of the three gauges overlapped. Although Rheem Creek has had just 25-30% the total flow of the other creeks, its drainage area is actually only 15-17% the size. As seen in Figure 26, Rheem Creek has had about 1 cfs of average annual flow per square mile of drainage area in the watershed, while neighboring creeks have had between 0.5 and 0.6 cfs of average annual flow per square mile of drainage area.

How can we explain the differences in runoff between these watersheds that have very similar precipitation patterns? Most likely, Rheem Creek has more flow in proportion to its watershed size because of its higher percentage of impervious surfaces, from which rainfall runs off rather than infiltrating into the ground. While about 50% of the total Rheem Creek watershed is estimated to be impervious surfaces, only 15-20% of the Wildcat and Pinole watersheds are impervious (Contra Costa County Community Development Department and the Contra Costa Watershed Forum 2003). In short, the neighboring watersheds contain a greater proportion of undeveloped, unpaved land. Moreover, because most of the pervious surfaces in the Rheem Creek watershed are located below the site of the historical stream gauge, we can expect that the drainage area above the gauge had an even greater percentage of impervious surfaces than the watershed on average. Table 4 summarizes this information about these three creeks.

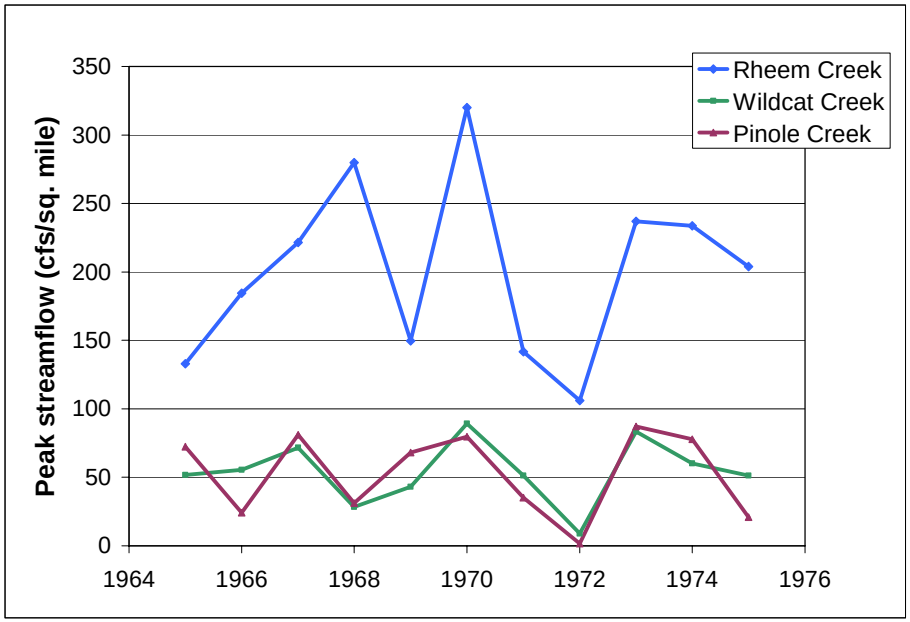
One way of determining if more impervious cover is the reason for the greater runoff in the Rheem Creek watershed would be to analyze the timing of runoff. Since the Rheem Creel watershed has more impervious cover, we would expect a greater proportion of the runoff to occur in the winter; in contrast, in the Pinole and Wildcat watersheds, we would expect more infiltration of rain into the ground and therefore less runoff during the winter.

Figure 27, which compares annual peak flows for the three creeks per unit drainage area, indicates that Rheem Creek has had a much higher peak flow per unit of drainage area.

Table 4: Comparison of Rheem, Wildcat, and Pinole Creeks

	Rheem Creek	Wildcat Creek	Pinole Creek
Land area draining to stream gauge (mi^2)	1.49	8.69	10
Average flow 1965-1974 (cfs) "Qav"	1.50	5.47	5.03
Qav per unit area (cfs/mi^2)	1.01	0.63	0.50
Estimated percent impervious	50%	20%	15%

Figure 27: Peak Annual Streamflow per Unit Drainage Area in Rheem, Wildcat, and Pinole Creeks (1965-1975)



Historical Streamflow in Rheem Creek

Human activities throughout the watershed have changed the timing and flow of water in Rheem Creek. Because streamflow records for Rheem Creek are only available for 1960-1990, a period which followed most of the watershed's development, there are no hydrologic data to quantify the extent to which urbanization and other land use patterns have changed the amount and timing of flow in the creek. However, based on a combination of hydrologic principles, understanding of the environmental setting and development history, and deductive reasoning, we can qualitatively describe how the hydrology has changed.

The rate of infiltration of rainfall is determined by several factors: land cover, slope, and soil type. Before extensive urbanization of the watershed in the twentieth century, the natural vegetation and uncompacted soils slowed runoff rates and increased infiltration of rainfall. In the middle and upper watershed where the land is more steeply sloping, runoff was more rapid than in the lower watershed. The different soils in the watershed have natural infiltration rates of between 0.06 and 0.6 inches per hour. Even before the watershed was paved, grazing in the watershed compacted the soil and lowered infiltration into the soil. With development, as more of the ground was covered by impervious surfaces such as buildings, paved roads, and parking lots, runoff increased and rainfall infiltration decreased, to as low as 0.01 inches per hour (Soil Conservation Service 1977).

In addition to an increase in the quantity of runoff, the timing of runoff also changed with development. Water travels more rapidly when it flows over paved surfaces than when it flows over vegetated areas. As the watershed developed, rainfall was transported into the stream channel more rapidly. In addition, storm drains, installed to rapidly remove water from urbanized areas to prevent flood damage, have changed the timing and amount of runoff and flow in the creek. Throughout the watershed, particularly in the middle and lower sections, storm drain networks move water rapidly from city streets into the channel.

The paving of the watershed and the construction of storm drains have resulted in higher flood peaks and a shorter lag time between the beginning of storms and an increased volume of water in the creek. Such an increase in storm runoff has many consequences in urban areas, including more frequent flooding and greater erosion (Dunne and Leopold 1978). It is possible that development of the upper watershed in the 1950s contributed to the flooding that occurred in the lower watershed in 1955. (1955 was only a slightly wetter year than average.)

The changes in Rheem Creek's hydrology probably provoked the alterations to the channel. The various channel improvement projects focused on rapid water evacuation through increased channel capacity and decreased sinuosity and roughness. The widening of the channel and encasement in concrete or trapezoidal channels created a straighter, smoother channel that moves water more quickly and is less likely to overtop its banks and flood adjacent communities. These projects also altered the channel hydrology: water now moves through Rheem Creek and into the Bay at a much quicker rate than it did in the nineteenth century.

While flooding in the middle watershed may have been addressed by the channel modifications, flooding has become a more serious problem in the upper watershed in recent years. Residents of the Rollingwood neighborhood in the upper watershed have reported serious flooding of streets and homes in recent years. In this case, there may be a number of factors contributing to the flooding, including: cattails and other invasive vegetation such as blackberry plants clogging the channel, a constricted and clogged culvert, and clogged storm drains (Bryant 2004). Debris, such as yard waste tossed into the creek, also seems to be contributing to the flooding (Davidson 2004). It is also likely that the flooding is exacerbated by development upstream that has led to increased siltation and an increased volume of water in the channel. The channel may not have adequate capacity to accommodate the amount of water that now flows in the creek (Hillard 2004). In this area, the Contra Costa County Flood Control District (CCCFCD) formerly maintained the channel by clearing it of brush and other debris, but in recent years the CCFCD has not

maintained the channel and flooding problems have worsened (Bryant 2004). To better understand the cause of the flooding, detailed hydrological studies of this reach of the creek will be necessary. Such studies have proven extremely challenging due to significant vegetation overgrowth and yellow jacket infestation in the creek corridor resulting from a long term lack of maintenance (Hillard 2004).

The Ecological Importance of Flooding

When creeks run through densely developed areas with houses built close to the banks, flooding is typically seen as an economic problem. Indeed, the many modifications to the Rheem Creek channel – removal of vegetation, straightening of the channel, and construction of rectangular and trapezoidal channels - were implemented to reduce the likelihood of regular floodplain inundation. However, flooding serves an important role ecologically for the species that live alongside the channel. Without regular inundation, floodplain species are deprived of the nutrients, moisture, and disturbance regimes that they require. Flood protection activities on Rheem Creek have transformed the channel from a dynamic living system to a static, confined, ecologically impoverished water conveyance structure.

Water Quality

An urban stream, Rheem Creek catches runoff from roads, rooftops, lawns, streets, parking lots, and driveways that flows directly into the channel or through storm drains that empty into the channel. Not surprisingly, this urban runoff is often quite contaminated, and it receives no treatment before entering the creek. Contamination occurs directly when pollutants are dumped into storm drain sewers or into the creek and indirectly when runoff picks up contaminants and carries them into the creek and storm drains. These pollutants include soap, concrete, paint, pesticides, and dirt.

Water contamination from dispersed and varied multiple sources like these is called nonpoint source pollution. Because Rheem Creek passes through several different cities on its short course to the bay, no single agency is responsible for the quality of water. While California is unique in having regulations that govern water quality from nonpoint sources, it is extremely difficult to estimate the total amount of nonpoint source pollution in a watershed and even more difficult to limit it through regulations. Collectively, the cities in the Rheem Creek watershed participate in the Contra Costa Clean Water Program; the main activities of the program are public education, inspection and enforcement, and industrial outreach.

Although Rheem Creek is not a drinking water supply source, the pollutants in the water do have harmful effects on plants and animals dependent upon the creek. Information about the water quality of runoff into Rheem Creek is limited to two seasons of data (water years 1995 and 1996) collected by the Contra Costa County Flood Control and Water Conservation District during storm events. Clearly, data about water quality is extremely limited. Although two years of data is not sufficient to generalize about water quality in Rheem Creek, it does allow us to identify potential problems.

The measured concentrations of substances in Rheem Creek water samples were compared to Water Quality Objectives (WQO) defined in the Water Quality Control Plan for the San Francisco Bay. WQOs are developed to achieve the highest level of water quality compatible with the ways a water body is used (California Regional Water Quality Control Board, San Francisco Bay Region 1995). For Rheem Creek, WQOs exist for cadmium, copper, lead, nickel, silver, and zinc to protect aquatic life and for mercury to protect human health due to the consumption of fish (Contra Costa County Flood Control and Water Conservation District, 1995 and 1996). Exceedance of a WQO means that there may be some impact of the particular substance to aquatic life. There are two types of WQOs. Acute WQOs protect organisms from acute toxicity, the detrimental effects of brief (96 hours or less) and continuous exposure to toxic substances. Chronic WQOs are protect organisms from chronic toxicity, the negative effects from long-term (greater than 96 hours) exposure to toxic substances, which are usually nonfatal (California Regional Water Quality Control Board, San Francisco Bay Region 1995).

The water quality test results indicated that runoff into Rheem Creek contained a number of substances at levels that are harmful to aquatic life. The data suggest that there are problems with zinc and lead in Rheem Creek. Mercury was also found at levels that may be harmful to human health. In addition, diazinon, a widely-used insecticide that is very poisonous to birds and aquatic organisms such as insect larvae and crustaceans, was detected in all of the samples taken, sometimes at toxic concentrations (Contra Costa County Flood Control and Water Conservation District 1995 and 1996):

While these results are evidence that the water in Rheem Creek may be lethal to aquatic organisms, the concentrations of heavy metals and pesticides found in the creek are actually typical of urban runoff from residential and commercial watersheds with some industrial activities (Contra Costa County Flood Control and Water Conservation District 1995 and 1996). The nearby creeks - Wildcat, San Pablo, and Pinole Creeks - are all listed as "Impaired Water Bodies" by California's water quality board, and diazinon is the "Pollutant of Concern" for all of these streams. In short, the levels of contamination in Rheem Creek not exceptionally high when compared to other watersheds with a similar mix of land uses.

Changes in Water Quality

The water quality in Rheem Creek has been affected by human activities throughout the watershed. As land uses in the watershed changed, the water quality in the creek deteriorated gradually. Creek water was probably safe for human consumption in the mid-1700s. With the introduction of cattle and agriculture during the Rancho period, water quality began to decline due to runoff containing chemicals used in farming and microbes in animal waste.

As the watershed developed, increasing amounts of toxic substances were transported to the creek. Runoff moving over streets and parking lots collects and concentrates pollutants from human activities, referred to as "non-point sources" of pollution. Non-point sources including lawn chemicals, oil, detergents, and pet wastes are all transported by rainfall runoff into the channel. The increase in impervious surfaces also affects water quality. Pervious surfaces that used to filter water are now greatly reduced throughout the watershed, so more toxic substances are more likely to be carried into the creek. The loss of riparian habitat, coupled with the lining of the concrete with packed earth and concrete, means that the creek has lost its capacity to filter and buffer contaminants that enter the water.

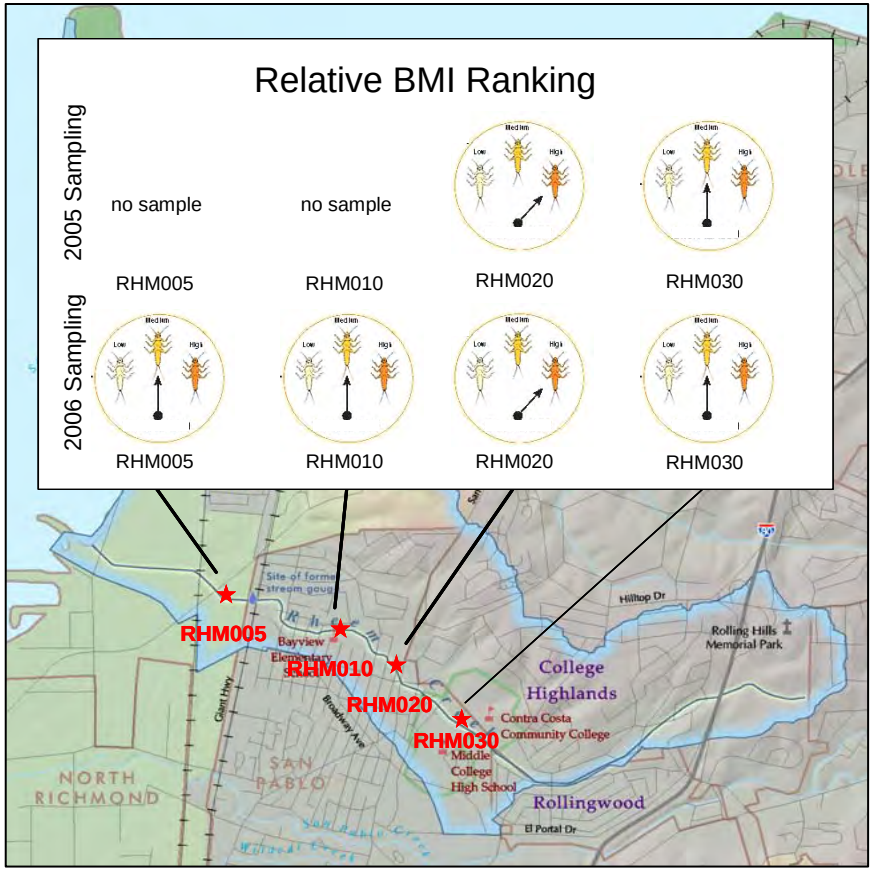
Since the 1940s and 1950s, when roads and housing developments were built throughout the middle and upper watershed as well as the headwaters, the creek has been a conduit for pollutants from lawns, driveways, and roads. In the lower watershed, tidal marsh once acted as a natural sponge, cleaning the water. Because the creek now runs in a riprap channel, the water is no longer filtered before it enters the San Pablo Bay. As a result, polluted water enters the San Pablo Bay, and tidal action distributes these contaminants. The San Pablo Bay is currently designated as an impaired waterbody; pollutants of concern include copper, mercury, nickel, selenium, diazinon, PCBs, chlordane, DDT, dieldrin, dioxin compounds and furan compounds (San Pablo Bay Watershed Restoration Program 2000).

Aquatic Life – Benthic Marco Invertebrates

In the spring of 2005 and 2006, the Natural Heritage Institute, its project partners, and volunteers completed benthic macro-invertebrate (BMI) monitoring along Rheem Creek. BMIs (aquatic bugs and other similar life forms that live in the creek) are good indicators of creek health. They respond to changes in water quality and other environmental factors in the watershed. Certain BMIs are more sensitive to pollution than others. If BMI monitoring reveals many more sensitive species, it is a good sign that the creek is healthy.

In 2005, the team monitored two complete sites in the upper and middle watershed. In 2006, the team, with the assistance of the City of Richmond, monitored four sites in the watershed; the two complete sites from 2005 and two additional sites further downstream (Figure 28). All monitoring was done with the

Figure 28: Relative BMI Ranking in Rheem Creek Watershed



assistance of Contra Costa County Community Development Department and consistent with the standard monitoring protocols of their Volunteer Creek Monitoring Program.

In both years, one or two sensitive taxa were found at each site. However, the overwhelming majority of BMIs found at each site were less sensitive and pollution tolerant. This indicates less than ideal water quality conditions in Rheem Creek.

Surprisingly, in both years, the sampling site with the best results was at Wanlass Park in the middle watershed (RHM020). This is surprising because the channel at Wanlass Park is an un-shaded, trapezoidal channel that is immediately downstream from a long stretch of creek that is underground. The reach at Contra Costa College (RHM030) is shaded and verdant (Figure 18) and would appear to be healthier, but BMI results indicate that it is not. While in the field, the team speculated that the Wanlass Reach was healthier because it had more brushy vegetation close to the channel and was rockier (broken concrete in the channel) that provided better habitat for BMIs.

Based on the limited data available, Rheem Creek's BMI results are comparable to those of San Pablo Creek, immediately to the south. San Pablo Creek is a much larger creek, yet subject to many of the same urban stressors that impact Rheem Creek. Other watersheds in Contra Costa County, with less urban development and more open space, tend to have more pollution sensitive species indicating better creek health.

IV. Community Vision

In an effort to begin to address watershed issues described above and to improve watershed health, between January 2005 and August 2006, the Rheem Creek project held a series of community visioning events. The overall goal of these events was to have the local community learn about the current state of Rheem Creek and its watershed, consider possibilities of what the creek and its watershed could look like in the future, and develop a concrete vision for the creek and its watershed.

To reach these goals, the project developed three events which built on each other, sequentially. Each event included experiential learning as well as time for reflection and visioning. The first event was a tour of the Rheem Creek watershed. The goal was to expose people to the current state of the watershed, elicit local knowledge of current and past conditions, and begin thinking about the future. The second event was a tour of Marin County's China Camp State Park, located directly across the San Pablo Bay. The goal of this event was to expose community members to a watershed that is physically similar to Rheem Creek, but is in a more natural state. At this meeting, community members turned their attention to visioning for the future of Rheem creek and its watershed. The third event aimed to present the community visions and restoration designs that had emerged from pervious meetings, as well as develop buy-in for continued work in the watershed. The following sections highlight each event with more details.

First Community Event: Watershed Tour, January 2005

The goal of the first community event was to expose people to the current state of the watershed, elicit local knowledge of current conditions, and begin thinking about the future. At this event over seventy people (participants came from Parchester Village and North Richmond, as well as from elsewhere in Richmond, Berkeley, Oakland and other nearby cities) boarded a bus, and went on the first-ever Rheem Creek Watershed Tour. The tour began at the headwaters up at the Rolling Hills cemetery. While the adults listened to short lectures on environmental justice and watershed science, some of the younger participants ventured down to where you can first see Rheem Creek's waters.

Moving downstream to the upper portion of the watershed, we visited the Rollingwood neighborhood, where resident and community organizer Marty Gomez spoke to the group about the problems with flooding that he and his neighbors experience. The tour stopped briefly at the Contra Costa Community College, to view an opportunity for future creek restoration where students can study nature and enjoy its beauty. At the mouth of the creek the group walked along the lower watershed, learning about the community's historic fight for keeping the lower portion of Rheem Creek a vibrant and healthy open space.

Figure 29: The Rheem Creek Watershed Tour

At the Parchester Village Community Center, participants broke up into small groups to answer two main questions: 1) What did you see today? 2) What would you want to change? The elders of the communitymet in a separate group and focused on how the watershed has changed over their lifetime. Finallythe full group discussed what the community as a whole wants. The results are included below.

Figure 30: Break-out Group Discussion of the Rheem Creek Watershed

Positive observations:

- Some native wildlife
- The beauty and aesthetic value of the shoreline and marsh area
- The community awareness and activism that is taking place
- The value of the creek and open space

Less positive observations:

- A general lack of wildlife and native vegetation.
- A lot of invasive vegetation, and not much native creek vegetation
- Often hard and sometimes impossible to see the creek
- Loose dirt at the headwaters and potential for creek sedimentation
- Use of pesticides and fertilizers
- Uncertain of future shoreline/lower watershed development activity
- Trash in and along creek
- Creek is often used as a dump by local residents

Changes seen by the community elders:

- The creek has narrowed and contains less water
- There has been an increase in trash and noise
- There has been an increase in new developments
- Earthquakes could threaten communities along the shoreline/lower watershed

What the community wants:

- Educational opportunities for everyone so people know how precious the watershed is
- Community participation where citizen groups help monitor and keep the creek clean and healthy
- Better management of the creek and delineation of tasks to keep it clean
- Keep the shoreline/lower watershed as open space and use it for recreation and learning
- Keep an eye on development
- Have the creek and its surroundings to be a community resource, which includes restoring Rheem Creek

Second Community Event: Trip to China Camp State Park, June 2005

At the second community event the community became environmental planners. The goal was for community members to consider a potential future for Rheem Creek and begin visioning. Local community members traveled directly across the San Pablo Bay to Marin County's China Camp State Park. There, the group visited a neighboring watershed whose shoreline provides a look at what Rheem Creek and its shoreline might have looked like hundreds of years ago. Listening to a presentation by plant ecologist Dr. Phyllis Faber, the group learned about the importance of salt marshes, and the role of local plants and wildlife.

After this part of the tour, the group took part in a visioning exercise for three parts of the Rheem Creek watershed: the lower watershed, where

Rheem Creek meets the Bay, the City of San Pablo reach including Bayview Elementary School, and the creek at Rollingwood and Contra Costa College. The group created maps that helped document the community's vision for the creek and its surroundings.

Figure 31: Rheem Creek Community Members at China Camp

For example, in maps of the lower watershed, where Rheem creek meets San Pablo bay, one group noted that it wanted the creek restored to natural conditions, and that there should be a bay trail and an overpass so that local residents can access the trail. A second group noted that it wanted the Breuner site (through which Rheem Creek passes) to look more like China Camp. Putting these maps together, landscape architects from the Restoration Design Group produced a conceptual restoration plan for the three parts of the watershed (See Chapter V. Restoration Opportunities).

For the middle portion of the watershed, near Bayview Elementary school, residents thought about restoring the creek and making it more accessible to local school children. In the upper-middle portion of the watershed, Rollingwood residents considered restoring meanders to the creek.

These maps have been used directly and indirectly to inform professional restoration designs of the creek that will hopefully be used in future restoration efforts.

Third Community Event: Finailizing Community Visions and the Watershed Declaration, August 2006

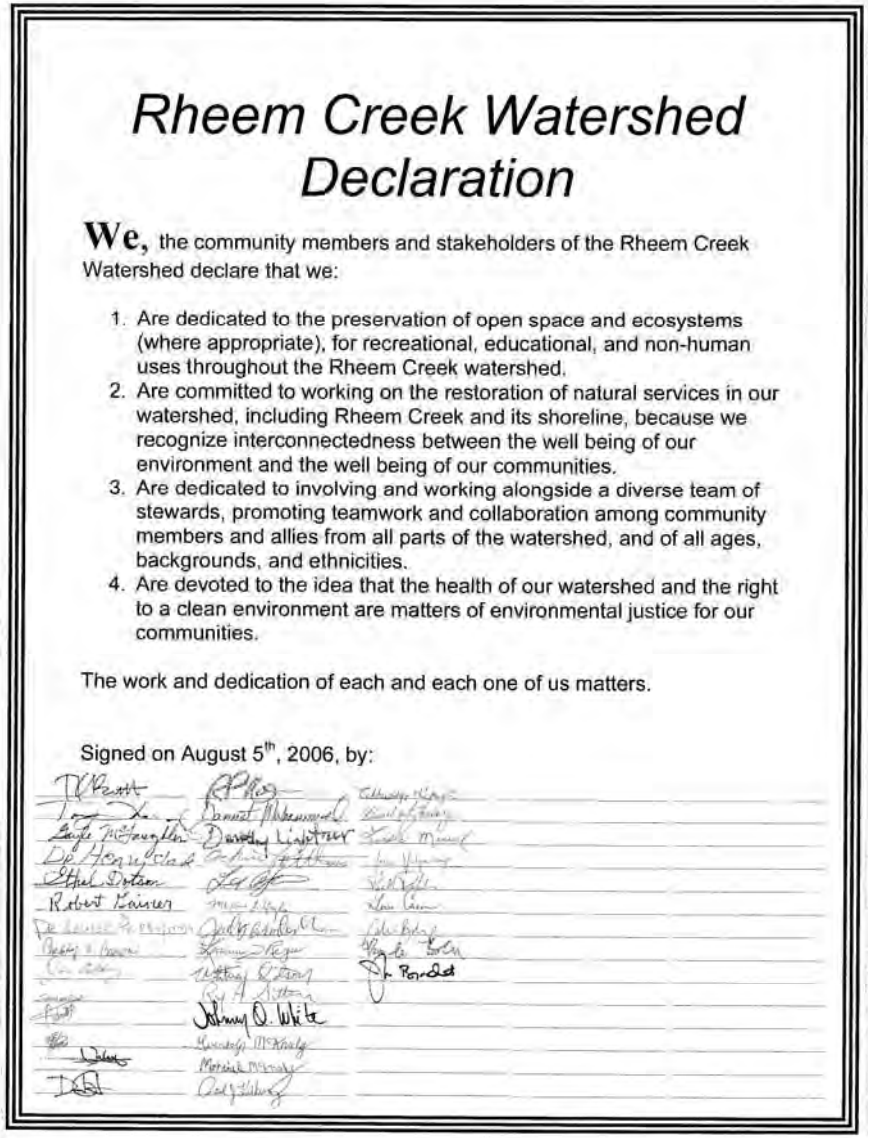
The third Rheem Creek community event aimed to present the community visions that had emerged from pervious meetings, and develop buy-in for continued work in the watershed. Roughly fifty local community members and elected officials attended this meeting.

At this meeting, the group viewed a Rheem Creek slideshow, which Mohania McKneely, the Project's youth intern, narrated. The slideshow reminded people of all that had already been accomplished and learned. After the slideshow, residents attended one of the three design charettes that they had worked on previously so as to comment on and finalize the desings. With the maps from the second event incorporated into professional conceptual design renderings, the community was able to observe the results of its visioning work, and make additional comments. Each group was asked to report its comments to the whole group for a thought-provoking exchange of ideas about the conceptual designs.

To close, the project team presented the Rheem Creek Watershed Declaration, a document that was meant to elicit a commitment to continued work in the watershed. All participants were asked to sign. Today, the document has traveled to numerous events, reminding elected officials, groups working along the North Richmond Shoreline of the values espoused by the stewards of Rheem Creek.

Overall, the community events were incredibly successful. A growing number of people were able to experience the watershed and Rheem creek first-hand. A fairly consistent group of local community members was able to document its vision for Rheem Creek throughout the watershed. As a result, the final conceptual designs have formed a foundation that can inform future planning efforts in the watershed. Equally as important, the events developed a larger group of concerned citizens that are more aware of their role as local stewards.

Figure 32: The Rheem Creek Watershed Declaration



Rheem Creek Watershed Declaration

We, the community members and stakeholders of the Rheem Creek Watershed declare that we:

1. Are dedicated to the preservation of open space and ecosystems (where appropriate), for recreational, educational, and non-human uses throughout the Rheem Creek watershed.
2. Are committed to working on the restoration of natural services in our watershed, including Rheem Creek and its shoreline, because we recognize interconnectedness between the well being of our environment and the well being of our communities.
3. Are dedicated to involving and working alongside a diverse team of stewards, promoting teamwork and collaboration among community members and allies from all parts of the watershed, and of all ages, backgrounds, and ethnicities.
4. Are devoted to the idea that the health of our watershed and the right to a clean environment are matters of environmental justice for our communities.

The work and dedication of each and each one of us matters.

Signed on August 5th, 2006, by:

V. RESTORATION OPPORTUNITIES

Chapters 1-3 emphasized the extent to which human activities have affected Rheem Creek and its watershed. Changing land uses over the past two centuries have led to the degradation of the natural environment throughout the watershed, especially the loss of ecologically important baylands habitats. Over the past sixty years, residential developments encroaching on the channel have eliminated most riparian habitat and led to channel modifications to protect communities from flooding.

Development throughout the watershed has altered the timing and amount of water in the channel as well as the quality of water. The creek has lost much of its natural capacity to filter pollutants. This urbanization has also had an effect on the human community of the watershed, which now has extremely limited opportunities to visit aesthetic open spaces.

But as Chapter 4 demonstrated, the story of Rheem Creek does not end here. Although Rheem Creek will never be restored to its pristine eighteenth century conditions, substantial opportunities exist to enhance and improve the health of the creek, transforming it into an aesthetic, educational, and ecological amenity for the human and non-human communities of the watershed. The community idenfied many of these opportunities through its visioning process. This chapter summarizes some of these opportunities: projects already underway, projects that have been proposed, and additional recommendations. Figure 33 locates each opportunity on the watershed map.

Restoration and Enhancement

Upper Watershed

As identified in the community visioning process, there is an opportunity to restore and enhance the channel where Rheem Creek flows through Contra Costa College. In this reach, the protential exists to restore 1,000 feet of Rheem Creek and to daylight (uncover) 100 feet of a tributary (Figure 34). The restoration would include the replacement of non-native plants with native species. Such a project would alleviate flooding, create natural habitat for birds and fish species, reduce erosion of the stream banks, and improve water quality by adding vegetation able to filter urban runoff. (Bryant 2004).

There are social benefits to the project as well. This restoration project presents an opportunity to educate and involve college students and faculty as well as community members in restoration and stream monitoring programs. College students and the 260 nontraditional high school students onsite can study the creek through classes in chemistry, biology, and environmental studies and even participate in the restoration project (Bryant 2004). In short, there is great potential to transform the creek into an aesthetic, recreational, and educational resource for the Contra Costa College campus community and neighbors.

Another opportunity in the upper watershed can arise from what is currently a problem: flooding in the Rollingwood neighborhood. Residents have recently organized an effort to address flooding and began to work with local agencies to seek a long-term solution. Working together, residents and public agencies can improve the health of the channel (and alleviate the flood problem) by working together to remove the debris and non-native plant species that are clogging the channel. Improvement of the creek in this reach will be an ongoing effort: residents will need to clear debris and non-native plants from the channel continually and keep their own yard waste out of the creek. In this reach, more information about the creek's hydrology – the timing and amount of water in the channel as well as the channel dimensions – will be essential to address the flooding issue.

Although the creek is viewed more as a nuisance than an amenity in the upper watershed, it is possible that the momentum generated from working together to resolve the flooding problems can be redirected to enhancement of the creek after the flooding problems are solved. Because the creek runs through private property, a publicly-funded restoration project is unlikely. However, as residents focus more attention on the

Figure 33: Major Opportunities for Restoration in the Rheem Creek Watershed

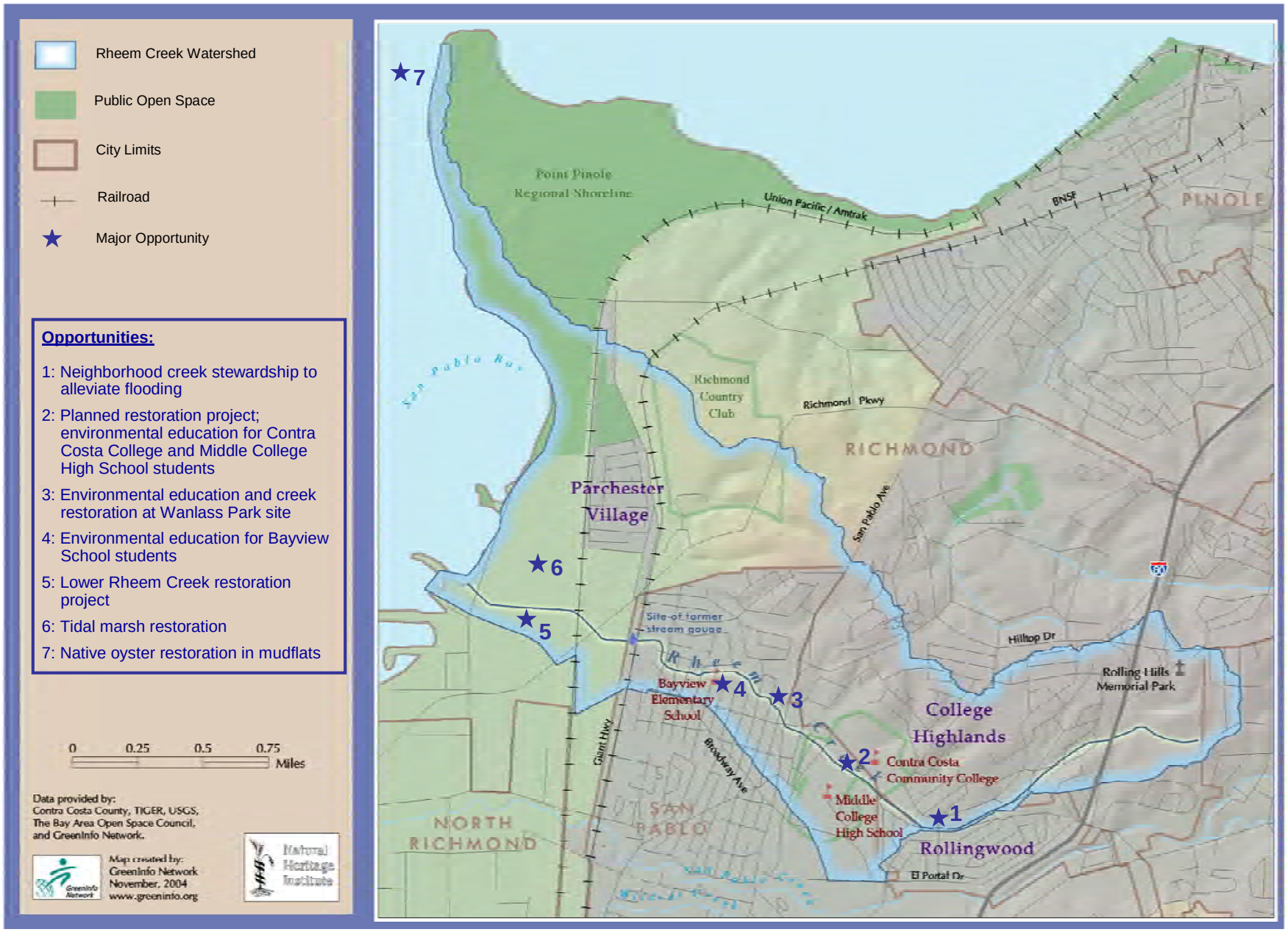


Figure 34: Conceptual Restoration Design for Contra Costa College Reach

RHEEM CREEK CONCEPTUAL RESTORATION PLAN

CONTRA COSTA COMMUNITY COLLEGE AT THE STUDENT UNION

5 MAY 2006

PROJECT MANAGER:
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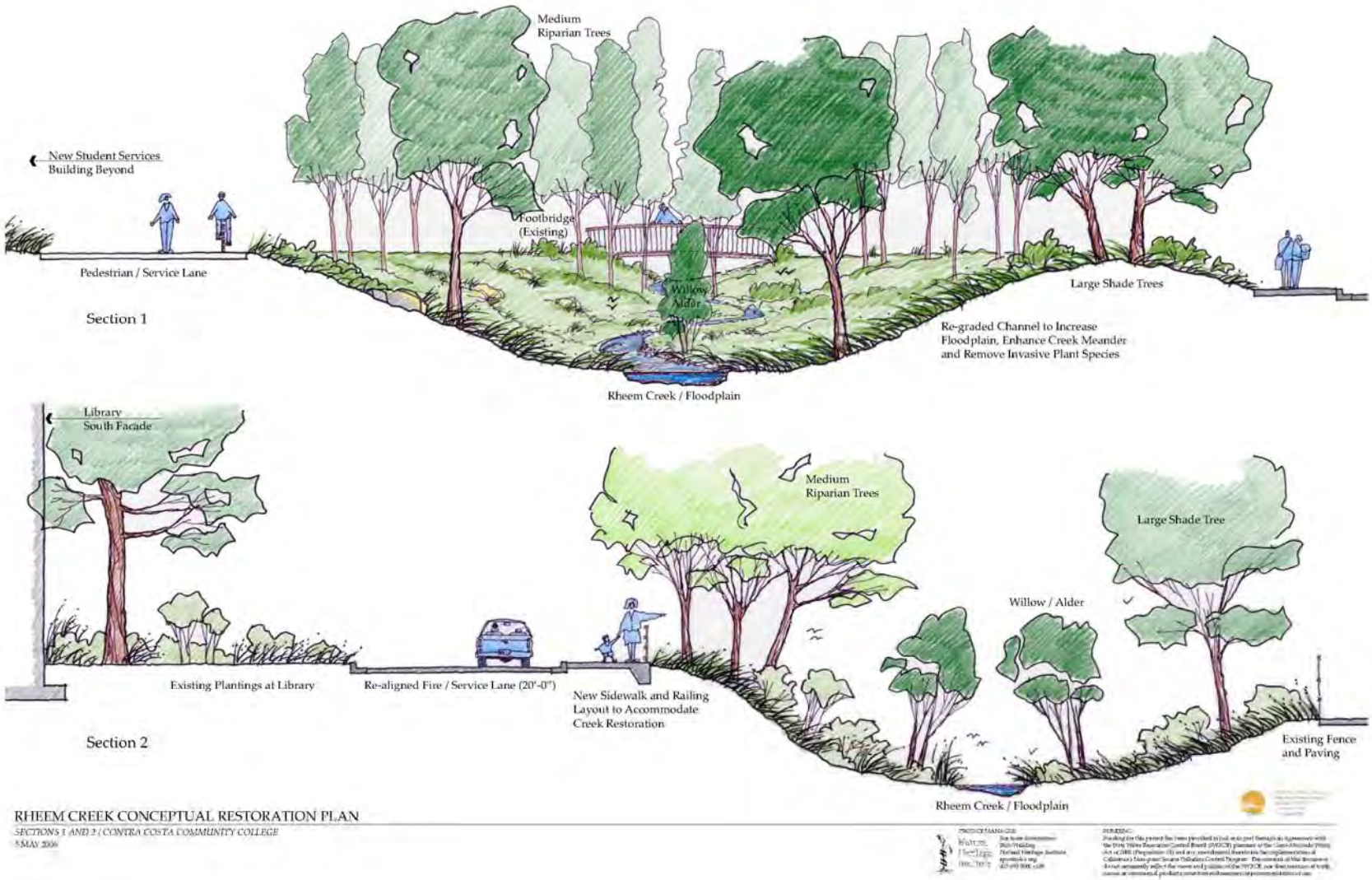
FUNDING
Funding for this project has been provided in full or in part through an Agreement with the State Water Resources Control Board (SWRCB) pursuant to the Costa-Machado Water Act of 2000 (Proposition 13) and any amendments thereto for the implementation of California's Non-point Source Pollution Control Program. The contents of this document do not necessarily reflect the views and policies of the SWRCB, nor does mention of trade names or commercial products constitute endorsement or recommendation of use.

Figure 34 (cont.): Conceptual Restoration Design for Contra Costa College Reach



creek and see restoration projects in other areas, they may become interested in enhancing its appearance and health. Simple actions include ensuring that the creek stays clean and even planting riparian trees and shrubs alongside its banks.

Middle Watershed

In the middle section of the watershed, the City of San Pablo is planning a creek restoration project as part of a new six acre city park, Wanlass Park. In partnership with the Contra Costa County Flood Control and Water Conservation District, the City of San Pablo plans to restore about 450 linear feet of Rheem Creek from its outflow at San Pablo Avenue and Rivers Street down to 20th Street. The restoration will involve planting riparian trees, shrubs, and grasses to cover most of the creek bank on both sides; the goals are to restore a portion of the riparian corridor and create a semi-shaded canopy over the creek. The City hopes to develop an educational center at Wanlass Park. The park may also include a gate that will enable students from Bayview Elementary School to enter the park and visit the creek (El-Telbany 2004).

Wanlass Park will be the second park in all of San Pablo and the only park in the northern portion of the city. In addition to providing the community with access to a restored reach of Rheem Creek, it will also serve as a much-needed green space in this densely developed community (El-Telbany 2004).

Below the site of Wanlass Park, Rheem Creek passes through the yard of the Bayview Elementary School in the West Contra Costa Unified School District. Here the creek has the potential to become a unique on-site educational resource. A creek enhancement or restoration project, such as the creation of a more natural channel and the planting of native species alongside the creek, can transform the creek into an outdoor science classroom for the school (Figure 35). One example of a successful school site restoration project is on Blackberry Creek in Berkeley. In the schoolyard of the Thousand Oaks Elementary School, a 250 foot-long reach of Blackberry Creek was daylighted, planted with native species, and turned into a small park area in 1995.

Lower Watershed

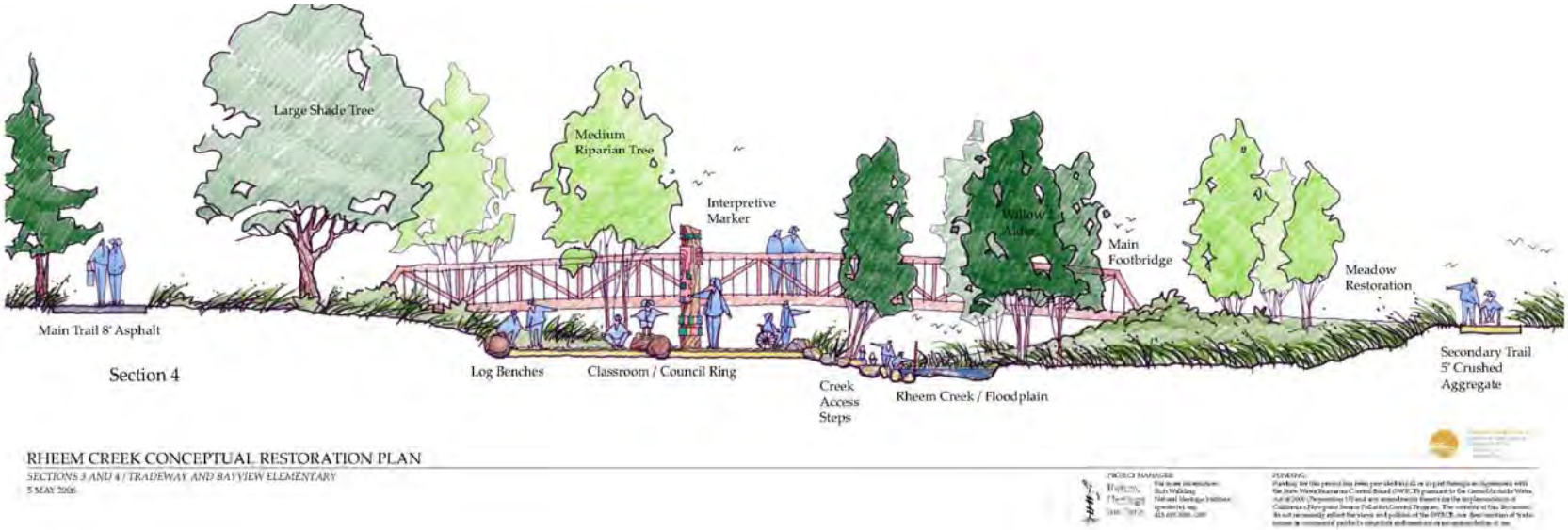
The lower watershed lands are ecologically and socially important, and therefore have great potential for restoration and enhancement. The San Francisco Bay Area Wetlands Ecosystem Goals Project developed specific habitat recommendations toward the goal of re-creating and sustaining diverse, healthy communities of fish and wildlife in the baylands. For western Contra Costa County, the Goals Project (1999) recommends the following:

- Protection and enhancement of existing tidal marshes.
- Protection and restoration of tidal marsh south of the Point Pinole Regional Shoreline at the Breuner Property, and connection to Giant Marsh.
- Restoration of vernal pools in the adjacent uplands.

The Goals Project (1999) concluded: "Implementing these recommendations would improve habitat conditions for tidal marsh-dependent small mammals, such as the salt marsh harvest mouse. Many species of Bay fishes that use tidal marsh habitat also would benefit from any tidal marsh improvements in this segment…Restoring and improving high marsh/upland transitions would benefit populations of several rare plants." A recent study of the marsh in the lower watershed concurred, finding that "with the restoration of Breuner Marsh and the ensuing recolonization of the area with the appropriate tidal marsh vegetation… restoration will provide a continuous band of high-quality habitat on a larger reach of shoreline" (Live Oak Associates, Inc. 2001).

Figure 35: Conceptual Restoration Design for Bayview Elementary School Reach

Figure 35 (cont.): Conceptual Restoration Design for Bayview Elementary School Reach



A major restoration project is possible on the lower reach of Rheem Creek at Breuner Marsh (Figure 36), where the East Bay Regional Park District has recently acquired several hundred acres. The potential exists to transform 2,400 linear feet of Rheem Creek from a trapezoidal earthen flood control channel into a more natural, meandering creek with greater wildlife value. This project could include planting native riparian species along the creek bank to replace the non-native species and restoring the upland areas to a more native state.

Community Education and Involvement

As the creek is restored in different places, more opportunities will exist for watershed residents to learn about and interact with the creek. Increased human enjoyment of the creek is likely to lead to greater interest in learning about the creek and its watershed and getting involved in restoring and enhancing these resources.

Opportunities for watershed residents to get involved in determining the future of the Rheem Creek watershed are abundant. Efforts to educate and involve watershed residents in future creek monitoring and management decision-making are underway.

As watershed residents organize themselves, they may choose from a number of possibilities for future projects and activities. These include developing and overseeing planned and desired restoration projects, advocating for preservation of important watershed lands, and developing watershed and creek education programs for youth and adults. The specific activities are best determined by local residents. It is our hope that this report will provide the necessary background information for residents to prioritize among the many possibilities, including those not listed here.

The creek also has the potential to become a valuable on-site educational resource for the community. Students attending Bayview Elementary School in the middle watershed and Contra Costa College can learn hands-on about riparian ecology, hydrology, and water quality.

Figure 36: Conceptual Restoration Design for Lower Rheem Creek

Watershed residents can help to fill gaps in our understanding of the creek by collecting information about water quality and aquatic life in the channel through a citizen monitoring program.

Pollution Reduction

As noted earlier, the boundaries of the Rheem Creek watershed do not follow any political boundaries, and the creek runs through several different cities and unincorporated neighborhoods on its path to the bay. Because most of the pollution in Rheem Creek comes from nonpoint sources, it is unlikely that regulations – which are typically directed at larger industrial "point" sources – will improve the water quality of the creek. Therefore, any improvements in water quality are likely to come from the many individual actions of watershed residents, not government agencies. Simple actions can help protect the quality of water in Rheem Creek and also reduce flooding in local neighborhoods. These include:

- keeping yard waste such as grass clippings and leaves out of storm drains and creeks
- leaving grass clippings on the lawn
- reducing or eliminating the use of pesticides and herbicides on lawns and gardens
- using less toxic alternatives to pesticides and herbicides
- checking for and fixing oil leaks on cars
- keeping oil off of driveways, streets, and the ground and out of storm drains
- recycling used motor oil, antifreeze, filters and other automobile fluids
- cleaning cars in areas that are easy to clean up and where soapy water will not run into storm drains
- disposing of soapy water in sinks rather than storm drains

The Contra Costa Clean Water Program provides information about how that watershed residents and businesses can improve the water quality in Rheem Creek.

Final Thoughts

The movement to restore and enhance Rheem Creek and its vital watershed lands has only just begun. Achieving the vision of a healthy, aesthetic creek that enriches the lives of people throughout the watershed will require new collaboration and leadership. Most importantly, the future of Rheem Creek depends upon residents from all parts of the watershed coming forward to advocate and work together, along with local city and county governments, for a healthier creek and protected watershed lands. We hope that this report can provide local residents and officials with the information and analysis necessary to develop a long-term vision and plan for these irreplaceable natural resources.

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BASELINE MITIGATION AND MONITORING REPORT 2013

FOR THE

PARKWAY COMMERCE CENTER PROPERTY

CONTRA COSTA COUNTY, CALIFORNIA

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January 2013

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ATTACHMENT 3 CORPS OF ENGINEERS DATA SHEETS

This report should be cited as: Olberding Environmental, Inc. January 2013. *Baseline Survey Monitoring (2013) Wetland Mitigation & Monitoring Report for the Parkway Commerce Center Property, Contra Costa County, California*. Prepared for Capital Partners Development Company, LLC, Rancho Cordova, California.

1.0 PROJECT OVERVIEW

The Parkway Commerce Center Property is located on a one-acre site located between the Santa Fe Railroad and Southern Pacific Railroad tracks and directly beneath Richmond Parkway within the City of Richmond, Contra Costa County, California. (Attachment 1, Figures 4). The Parkway Commerce Center Property is located west of Giant Road and the Santa Fe Railroad tracks, east of the Southern Pacific Railroad tracks, north of Parr Avenue and south of Morton Avenue at the terminus of Collins Way (Attachment 1, Figure 2). Attachment 1, Figure 3 illustrates the location of the property on the Richmond 7.5 minute USGS quadrangle map. Attachment 1, Figure 4 contains an aerial photograph of the Property. The site is accessed by taking the Giant Road exit off Richmond Parkway, traveling north to the Collins Avenue intersection. At Collins Avenue, proceed south to the property.

The mitigation on this Property offsets the impacts to a total of 0.814 acres of which 0.793 acres is jurisdictional open water channel and seasonal wetland habitats and 0.121 acres is temporary impacts to seasonal wetland habitat associated with the proposed project. To mitigate project impacts, a higher functioning seasonal wetland habitat (0.93 acres) was created and restored. The on-site area designated for seasonal wetland mitigation is located in the northwest corner of the property adjacent to the Southern Pacific Railroad tracks (Attachment 1, Figure 5). In addition, up to 1.15 acres of vegetated riparian buffer habitat will be created along both sides of Rheem Creek. The buffer area is intended to protect the water quality of the Creek by removing suspended particulates intended to protect the water quality of the Creek by removing suspended particulates and dissolved and suspended contaminates transported by runoff from the surrounding uplands areas.

The mitigation involved the creation of seasonal wetland habitat in areas currently occupied by upland habitats. Next year, the vegetated riparian buffer habitat will be created as part of a flood terrace that would be graded along the creek channel. This terrace will be graded above the Corps jurisdictional boundary (ordinary high water) to accommodate periodic high flows. All mitigation components occur in the general vicinity of the impacted areas allowing for on-site and in-kind mitigation opportunities.

2.0 REQUIREMENTS

The mitigation monitoring plan calls for the evaluation of performance criteria (percent cover and site maintenance), and trend characteristics (plant vigor/health, and natural reproduction/ recruitment) for the mitigation site. Evaluation of these criteria will allow a determination of the need for erosion control, supplemental seeding, weed control, and identify any human

disturbance. Also, the evaluation will determine the success or need for improvement of the mitigation effort. Table 1 below provides the initial target functions for the mitigation site and indicates the status of each function as of December 27, 2012. Target function will be quantitatively evaluated by monitoring the percent cover of wetland plant species occurring at the mitigation site. The following conditions will be monitored to determine compliance as exhibited in Table 1.

Table 1
Parkway Commerce Center Project Mitigation Monitoring Objectives

Mitigation Site Target Function	Status 12/27/12
(1). Approximately 0.91 acres of seasonal wetlands will be created for the 0.79 acres of jurisdictional open water channel and regulated seasonal wetland habitats impacted. Additionally, 0.02 acres of seasonal wetlands will be restored following construction of the mitigation site;	Construction resulted in the creation and restoration of 0.93 acres of wetland habitat as of December 2012.
(2). The wetland mitigation area will be maintained in as natural a state as possible. It will not be watered, mowed, burned, grazed, or otherwise maintained, except as part of an approved long-term plan to maintain or enhance the natural values of the established habitats;	Trash was observed in the upland area of the wetland as of December 2012.
a. No construction would occur in areas under Corps jurisdiction outside the permitted project footprint;	No violation as of December 2012.
b. Unless authorized by the Corps, no work would be conducted in or around the created wetland areas that may modify the hydrologic regime of the created habitats once established;	No violation as of December 2012.
c. No farming, livestock grazing or plowing will be allowed;	No violation as of December 2012.
d. Off-road vehicle use would be restricted;	No violation as of December 2012.
e. No work would be conducted in or around the restored wetlands area that may modify the hydrologic regime of the wetlands once established; and	No violation as of December 2012.
(3) A 20-foot wide vegetated riparian buffer area will be established along each side of Rheem Creek as it flows though the project site (1.15 acres).	No buffer established as of December 2012. Planned for Spring 2013.

Table 2 provides the quantitative vegetation success criteria for each year's monitoring effort.

Table 2 **Quantitative Mitigation Success Criteria**
1. Absolute plant cover is at a minimum of 60% within two years and 80% within five years;
2. The site will be seeded with a native wetland plant seed mix;
3. Dominance will be determined using the Corps' methodology for identifying jurisdictional wetlands; and
4. The site will not include more than 10% cover by non-native invasive plant species.

The objectives of the Baseline monitoring report were to ensure that the required mitigation planting had been conducted in relation to the development project, identify maintenance activities needed, provide photo documentation points, identify remedial actions to be performed, and provide final conclusions on the establishment of riparian and wetland habitat within the mitigation site.

3.0 SUMMARY DATA AND RESULTS

The wetland mitigation site was completed in the fall of 2011. This included the creation of seasonal wetland habitat in areas that used to be occupied by upland habitats. The establishment of a vegetated riparian buffer along the entire length of Rheem Creek have yet to be constructed. Seven fixed photo points were established at the wetland site and are labelled P1-P7. There were supposed to be ten photo points, but due to technical difficulties, only 7 photos managed to be obtained. All future photo documentation will have 10 photo points. Photo-documentation of the site was conducted from these locations and will remain stationary throughout the five year monitoring period. In addition to the seven photo points, three transect locations were established. The transects are labelled T1-T3 and will include two transect data points each; one upland point and one wetland point. The transect locations were selected in order to accurately represent the developing wetland characteristics of the entire wetland mitigation site. The transect locations will also remain stationary throughout the five year monitoring period. At the time of the survey on December 27, 2012, the mitigation site was partially filled with standing water as a result of a recent raining period and the ground was very wet. The banks of the basin were also heavily vegetated by herbaceous plant species.

Current conditions of all features were documented in photographs taken from fixed locations surrounding the mitigation site. These photographs have been included in Attachment 2 of this report. During the December 2012 survey, wetland delineation data forms were completed in

accordance with Corps wetland delineation criteria identified in the 1987 "Corps Wetlands Delineation Manual" (1987 Manual)[1] and have been included in Attachment 3.

3.1 Performance Criteria

3.1.1 Percent Cover (Wetlands) and Percent Survival (Riparian)

Table 3
Wetland Mitigation Site Percent Cover Performance Criteria

Monitoring Year	Average Percent Cover of Obligate, Facultative Wetland, and Facultative Species
Year 0	30%

A total of 6 data points were monitored along the established transects at each of the data point locations during the Baseline survey. The data points located within the basins had both standing water and moist ground at the time of the survey. The banks of the basin were vegetated with a mix of upland and wetland plant species. Some emergent wetland vegetation was also observed within the basin. The total vegetative cover (native and non-native) within the wetland basin at the time of the Baseline survey is approximately 65% with the rest being standing water. Wetland determination data forms specific to the Arid West Region were completed for each data point and are included in Attachment 3.

3.1.2 Site Maintenance

Site maintenance observations were included in the performance criteria because of the critical importance of proper maintenance during the plant establishment period. Site maintenance was monitored to assess the adequacy of hydrological frequency and wetland plant establishment. The following site maintenance criteria were evaluated.

Trend characteristics to be monitored include natural recruitment, and plant health and vigor. The results of the trend characteristics monitoring will aid in the assessment of the site's progress. The trend characteristics of Plant Vigor and Health and Natural Reproduction/ Recruitment are described below.

[1] Environmental Laboratory. 1987. "Corps of Engineers Wetlands Delineation Manual." U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi. 100 pp. plus appendices.

3.1.3 Plant Health and Vigor

The health and vigor of each dominant native wetland species occurring in the wetland mitigation site will be qualitatively assessed. Riparian restoration stock will also be evaluated for health and vigor. The following factors were considered in the determination: plant color, seed development, new growth, herbivory, drought stress, fungal/insect infestation, and physical damage. Health and vigor of each dominant wetland species was rated as identified in Table 4.

Table 4
Plant Vigor and Health Ratings

Health and Vigor	Percent Healthy Foliage
High	67-100%
Medium	34-66%
Low	0-33%

The vegetation in the wetland mitigation area was mostly on moist upland soil consisting mainly of curly dock (*Rumex crispus*), bristly ox-tongue (*Picris echioides*), and grass(Poaceae); however emergent wetland vegetation is continuing to spread throughout the wetland. Other minor plants in the upland area were cheeseweed (*Malva parviflora*), Italian rye grass (*Lolium multiflorum*), creeping wild-rye (*Elymus triticoides*), mustard (*Brassica sp.*), Bird's-foot trefoil (*Lotus corniculatus*), and cranesbill (*Geranium sp.).* There was also vegetation submerged in the water , though this could be due to the recent rainfall and not a part of the natural hydrology of the wetland. This vegation consisted mostly of Hyssop loosestrife (*Lythrum hyssopifolia*), cranesbill (*Geranium sp.),* and burclover (*Medicago sp.).* Other minor plants in the submerged area were fennel (*Foeniculum vulgare*), Italian rye grass (*Lolium multiflorum*), bristly ox-tongue (*Picris echioides*), spiny cocklebur (*Xanthium spinosum*), common cocklebur (*Xanthium strumarium*), and umbrella sedge (*Cyperus sp.*). The trees and shrubs of the riparian buffer area have not been planted yet. The mitigation basin had standing water and the percent cover of vegetation (native and non-native) within the basin is approximately 65%. The health and vigor of the wetland vegetation will not be assessed now as the plants were just recently planted through hydroseeding and still need time to grow and evaluated to see if the process was successful or not. However, as of a December 2013 progress report, the health and vigor looks to be on target to be to be high in the future. The emergent vegetation is spreading however; and once the tree and shrub species of the riparian buffer area are planted, they will be closely monitored in the following years due to their recent installation and tendency of the basin to flood.

3.1.4 Abundance and Diversity

Abundance and diversity was assessed by qualitatively analyzing the total number of plant species observed at the wetland mitigation site. Vegetation on the banks consisted of species of cheeseweed (*Malva parviflora*), bristly ox-tongue (*Picris echioides*), mustard (*Brassica sp.*), Italian ryegrass (*Lolium multiflorum*), Creeping wild-rye (*Elymus triticoides*), Bird's-foot trefoil (*Lotus corniculatus*), and Cranesbill (*Geranium sp.)*. Vegetation within the basin consisted of Fennel (*Foeniculum vulgare*), Curly dock (*Rumex crispus*), Burclover (*Medicago sp.),* Cranesbill (*Geranium sp.),* Italian Rye grass (*Lolium multiflorum*), Bristly ox-tongue (*Picris echioides*), Spiny cocklebur (*Xanthium spinosum*), Common cocklebur (*Xanthium strumarium*), Umbrella sedge (*Cyperus sp.*), and Hyssop loosestrife (*Lythrum hyssopifolia*). This vegetation composition may change for both upland and wetland areas as hydroseeding takes more effect in the future.

3.1.5 Photo-documentation

A total of seven fixed photo points were established to document the wetland mitigation basins. The photos will be used to demonstrate the yearly conditions to which annual monitoring will be compared. The color photo plates are located in Attachment 2.

4.0 MAPS

See Attachment 1 for the USGS Quadrangle map, aerial photograph and wetland map with transect locations and photo points.

5.0 CONCLUSIONS

Success Criteria 1. ***Absolute plant cover is at a minimum of 60% within two years and 80% within five years.***
During the Baseline Survey monitoring event, the mitigation basin is currently not meeting the criteria of having a minimum of 60% wetland plant cover for the first 2 years. As this is a baseline survey and not the actual monitoring and the mitigation area was just created recently in the fall of 2012, the site still has time (two more years) to meet this goal. Currently, the absolute plant cover is approximately 30%.

Success Criteria 2. ***The site will be seeded with a native wetland plant seed mix.***
The site was hydroseeded with a native plant seed mix. As of December 2012, the native plants were starting to emerge, but there is still non-native herbaceous plants on site for them to

compete with. It will take some more time to see how effective the hydroseeding was on the plant diversity of the site.

Success Criteria 3. ***Dominance will be determined using the Corps' methodology for identifying jurisdictional wetlands.***
During the Baseline survey monitoring event, the majority of the vegetation within the wetland basin was slightly dominated by obligate species. Additionally, the vegetation on the banks of the basin was comprised of predominately facultative species.

Success Criteria 4. ***The site will not include more than 10% cover by non-native invasive plant species.***
There were quite a large number of non-native species growing in the basin and on the banks. Ox-tongue and Italian ryegrass are some of the dominant non-native species observed to be growing at the mitigation site, mostly in the upland areas. As of the December 2012 Baseline survey, non-native species composed of greater than 10 percent of total vegetation cover.

The results of the Baseline survey were mixed of both positive and negative aspects. Mitigation planting/seeding on the wetland had occurred and native wetland plants had established themselves in the wetland area. Native wetland plants have not yet reached 60% plant cover required after two years. However, since this is the Year 0 Baseline survey, there is still two years before the requirement needs to be met. The riparian buffer has not been created, so the riparian planting have not been done yet. Baseline survey for the riparian buffer area will be delayed to next year when the riparian buffer is planned to be created and mitigation planting done.

6.0 RECOMMENDATIONS

At the time of the Baseline survey monitoring, the basin was partially filled with standing water and had moist soil upland. While a majority of the plants were native species, there was still a large number of non-native species present. These non-native plants can affect native plant recruitment in future years. Currently, non-native vegetation cover is above the 10% in success criteria 4. Weeding is recommended to be done to bring down the number of non-native plants and to control their spread. No remedial planting/seeding is required at this time as most of the wetland plants seeded were still present and healthy and still spreading. As mentioned earlier in the report, the site was just recently hydroseeded, so it will take some time to see how effected it was in changing the plant diversity of the site. Several recommendations are advised to be done to ensure that the goals of the mitigation site will be met. First, as mentioned before, weeding should be considered to keep the non-native plants in check to prevent competition of resources

for the native plants. Second, trash needs to be removed periodically to prevent contamination of the wetland and prevent trash from taking away space that the native wetland plants can use to spread. If these recommendations are done, it is anticipated that the native wetland plants will increase its cover over time and the wetland mitigation will continue to trend toward the success criteria within the five year period.

ATTACHMENTS

ATTACHMENT 1
FIGURES

Figure 1 Regional Map
Figure 2 Vicinity Map
Figure 3 USGS Quadrangle Map for Richmond
Figure 4 Aerial Photograph
Figure 5 Transect Location Map of the Mitigation Area
Figure 6 Photo Points Map of the Mitigation Area

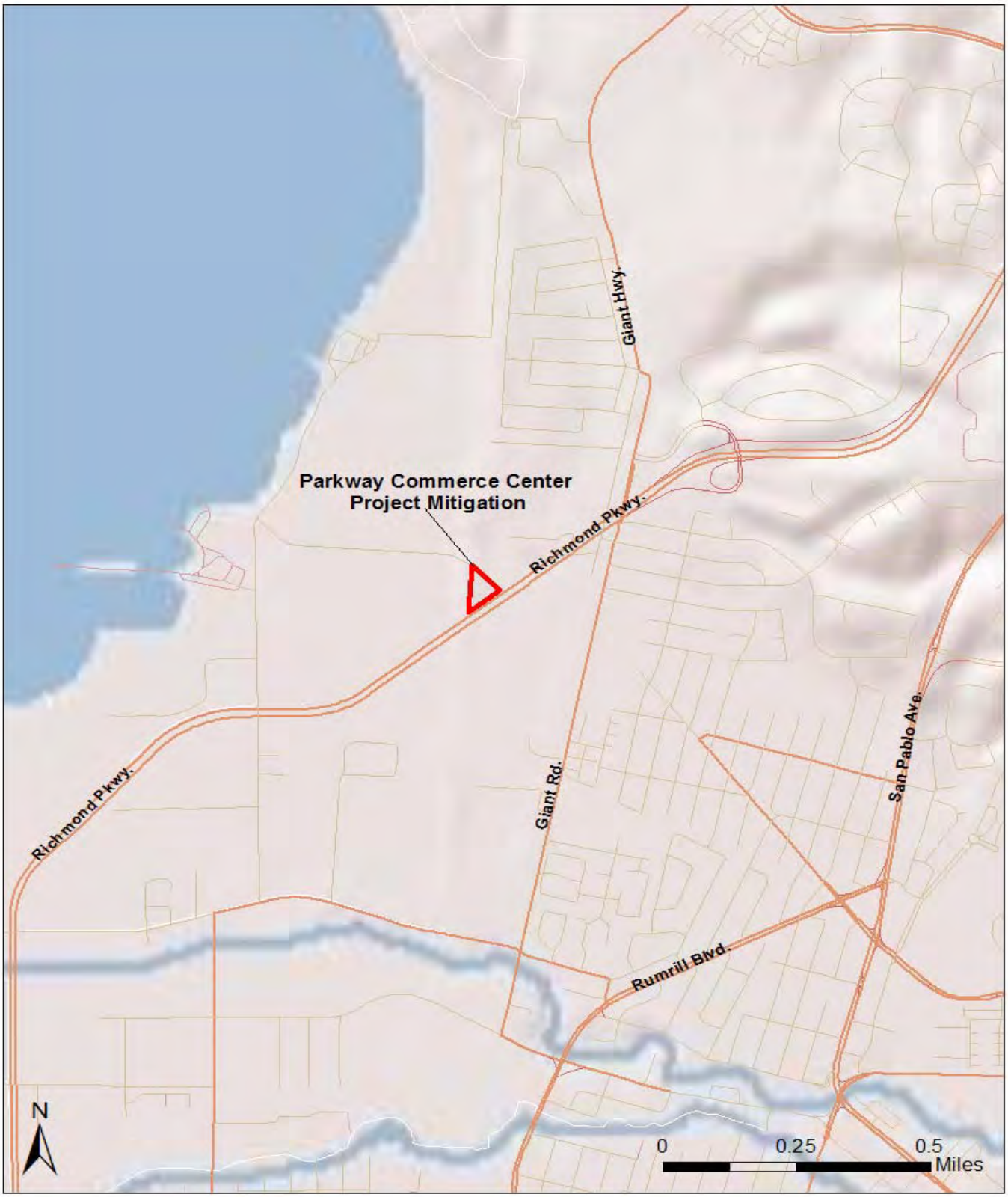
Figure 1
Regional Map

Olberding Environmental, Inc.
3170 Crow Canyon Place, Suite 260
San Ramon, California 94583
Phone: (925) 866-2111

This document is not intended for detail design work.

Figure 1
Regional Map of the Parkway Commerce Center Property
Richmond, California

Figure 2
Vicinity Map



Olberding Environmental, Inc.
3170 Crow Canyon Place, Suite 260
San Ramon, California 94583
Phone: (925) 866-2111

This document is not intended for detail design work.

Figure 2
Vicinity Map of the Parkway Commerce Center Property
Richmond, California

Figure 3
USGS Quadrangle Map for
Richmond

Olberding Environmental, Inc.
3170 Crow Canyon Place, Suite 260
San Ramon, California 94583
Phone: (925) 866-2111

This document is not intended for detail design work.

Figure 3
USGS Quadrangle Map of the Parkway Commerce Center Property
Richmond Quadrangle
Richmond, California

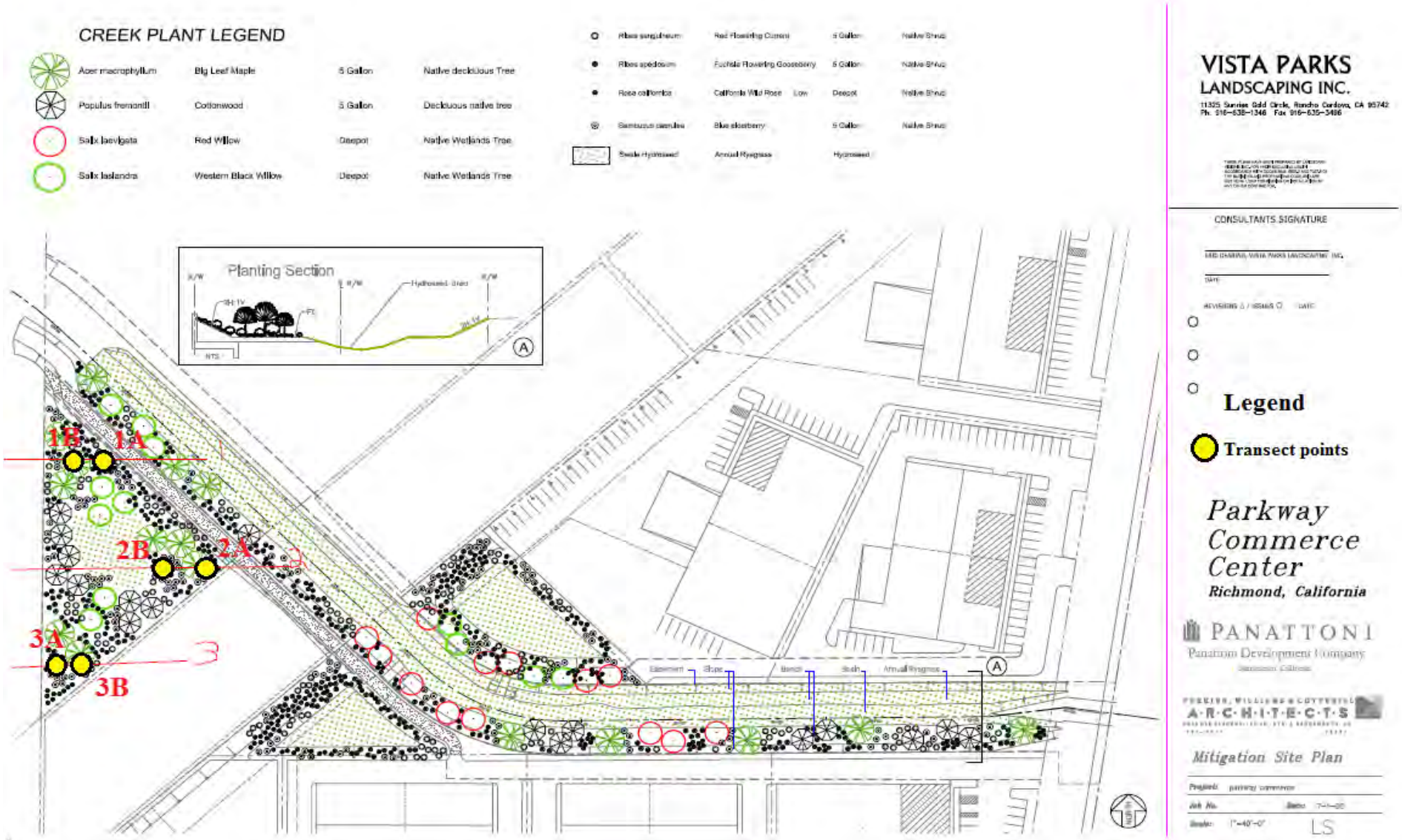
Figure 4
Aerial Photograph

Olberding Environmental, Inc.
3170 Crow Canyon Place, Suite 260
San Ramon, California 94583
Phone: (925) 866-2111

This document is not intended for detail design work.

Figure 4
Aerial Photograph of the Parkway Commerce Center Property
Richmond, California

Figure 5
Transect Location Map of the Mitigation Area



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San Ramon, California 94583
Phone: (925) 866-2111

This document is not intended for detail design work.

Figure 5
Transect Location Map of the Parkway Commerce Center Property
Richmond, California

Figure 6
Photo Points Map of the Mitigation Area

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San Ramon, California 94583
Phone: (925) 866-2111

This document is not intended for detail design work.

Figure 6
Photo Points Map of the Parkway Commerce Center Property
Richmond, California

ATTACHMENT 2
SITE PHOTOGRAPHS

Photo 1. View to the south of the wetland mitigation site.

Photo 2. View to the west of the wetland mitigation site.

Olberding Environmental, Inc.
Parkway Commerce Center Property – January 2013

Photo 3. View to the southwest of the mitigation site across Transect 1.

Photo 4. View to the northwest of the mitigation site across Transect 2.

Photo 5. View to the north of the mitigation site across Transect 1. Notice lots of non-native upland plants.

Photo 6. View to the east of the mitigation site across Transect 2. Also, notice the invasive Himalayan blackberries along the border of the site.

Photo 7. View to the south of the site from the northeast end of the property.

Olberding Environmental, Inc.
Parkway Commerce Center Property – January 2013

ATTACHMENT 3
CORPS OF ENGINEERS DATA SHEETS

WETLAND DETERMINATION DATA FORM – Arid West Region

Project Site: Parkway Commerce Center Property City/County: Richmond/Conta Costa Sampling Date: 12/28/12

Applicant/Owner: Panattoni Development Company, LLC State: CA Sampling Point: 1A

Investigator(s): Olberding Environmental Section, Township, Range: _____

Landform (hillslope, terrace, etc.): _____ Local relief (concave, convex, none): ____ Slope (%): 2

Subregion (LRR): _____ Lat: 37^ 58' 40.58" N Long: 122^ 21' 25.74" W Datum: _____

Soil Map Unit Name: Clear Lake Clay NWI classification: _____

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ☒ No ☐ (If no, explain in Remarks.)

Are Vegetation ☐, Soil ☐, or Hydrology ☐ significantly disturbed? Are "Normal Circumstances" present? Yes ☒ No ☐

Are Vegetation ☐, Soil ☐, or Hydrology ☐ naturally problematic? (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ☐	No ☒	
Hydric Soil Present?	Yes ☐	No ☒	**Is the Sampled Area within a Wetland?** **Yes ☐ No ☒**
Wetland Hydrology Present?	Yes ☐	No ☒	

Remarks: **Data point fails to meet all three Corps wetland criteria. Upland position for comparison to wetland point.**

VEGETATION – Use scientific names of plants.

Tree Stratum (Plot size:_____)	Absolute % Cover	Dominant Species?	Indicator Status
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Sapling/Shrub Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
5. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Herb Stratum (Plot size:_____)			
1. *Picris echioides*	20	yes	UPL
2. *Poaceae*	35	yes	UPL
3. *Rumex crispus*	30	yes	FAC
4. *Lolium multiflorum*	5	____	UPL
5. *Elymus triticoides and Malva parviflora.*	2	____	UPL
6. *Brassica sp*	1	____	FACU
7. *Lotus corniclatus*	2	____	FAC
8. *Geranium sp.*	5	____	FAC
50% = _____, 20% = _____	100	= Total Cover	
Woody Vine Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
% Bare Ground in Herb Stratum _____	% Cover of Biotic Crust _____		

Dominance Test Worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC:	1	(A)
Total Number of Dominant Species Across All Strata:	3	(B)
Percent of Dominant Species That Are OBL, FACW, or FAC:	**33**	(A/B)

Prevalence Index worksheet:

	Total % Cover of :	Multiply by:	
OBL species	0	x1 =	0
FACW species	0	x2 =	0
FAC species	37	x3 =	111
FACU species	1	x4 =	4
UPL species	62	x5 =	310
Column Totals:	100 (A)		425 (B)

Prevalence Index = B/A = 4.25

Hydrophytic Vegetation Indicators:

- ☐ Dominance Test is >50%
- ☐ Prevalence Index is ≤3.0[1]
- ☐ Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ☐ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? **Yes ☐ No ☒**

Remarks:

Project Site: Parkway Commerce Center Property

SOIL

Sampling Point: 1A

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (Moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-12	10 YR 3/3						Clay loam	

[1]Type: C= Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR C**)
- ☐ 1 cm Muck (A9) (**LRR D**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☐ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☐ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☐ Redox Depressions (F8)
- ☐ Vernal Pools (F9)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) **(LRR C)**
- ☐ 2 cm Muck (A10) **(LRR B)**
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ____

Depth (Inches): "

Hydric Soils Present? Yes ☐ No ☒

Remarks:

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1) **(Nonriverine)**
- ☐ Sediment Deposits (B2) **(Nonriverine)**
- ☐ Drift Deposits (B3) **(Nonriverine)**
- ☐ Surface Soil Cracks (B6)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Biotic Crust (B12)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☐ Oxidized Rhizospheres along Living Roots (C3)
- ☐ Presence of Reduced Iron (C4)
- ☐ Recent Iron Reduction in Tilled Soils (C6)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (2 or more required)

- ☐ Water Marks (B1) **(Riverine)**
- ☐ Sediment Deposits (B2) **(Riverine)**
- ☐ Drift Deposits (B3) **(Riverine)**
- ☐ Drainage Patterns (B10)
- ☐ Dry-Season Water Table (C2)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Shallow Aquitard (D3)
- ☐ FAC-Neutral Test (D5)

Field Observations:

Surface Water Present?	Yes	☐	No	☒	Depth (inches): ____
Water Table Present?	Yes	☐	No	☒	Depth (inches): ____
Saturation Present? (includes capillary fringe)	Yes	☐	No	☒	Depth (inches): ____

Wetland Hydrology Present? Yes ☐ No ☒

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Upland position for comparison to wetland. Data point located several feet up slope from topographic depression formed by swale.

WETLAND DETERMINATION DATA FORM – Arid West Region

Project Site: Parkway Commerce Center Property City/County: Richmond/Conta Costa Sampling Date: 12/28/12

Applicant/Owner: Panattoni Development Company, LLC State: CA Sampling Point: 1B

Investigator(s): Olberding Environmental Section, Township, Range: _____

Landform (hillslope, terrace, etc.): _____ Local relief (concave, convex, none): ____ Slope (%): 2

Subregion (LRR): _____ Lat: 37^ 58' 40.58" N Long: 122^ 21' 25.74" W Datum: _____

Soil Map Unit Name: Clear Lake Clay NWI classification: _____

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ☒ No ☐ (If no, explain in Remarks.)

Are Vegetation ☐, Soil ☐, or Hydrology ☐ significantly disturbed? Are "Normal Circumstances" present? Yes ☒ No ☐

Are Vegetation ☐, Soil ☐, or Hydrology ☐ naturally problematic? (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ☒	No ☐	
Hydric Soil Present?	Yes ☒	No ☐	**Is the Sampled Area within a Wetland?** **Yes ☒ No ☐**
Wetland Hydrology Present?	Yes ☒	No ☐	

Remarks: **The data point is positioned in a wetland. The three Corps criteria for wetlands are met.**

VEGETATION – Use scientific names of plants.

Tree Stratum (Plot size:_____)	Absolute % Cover	Dominant Species?	Indicator Status
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Sapling/Shrub Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
5. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Herb Stratum (Plot size:_____)			
1. *Rumex crispus*	1	____	FAC
2. *Medicago sp.*	5	____	FACU
3. *Geranium sp.*	5	____	FAC
4. *Cyperus sp.*	1	____	OBL
5. *Lythrum hyssopifolia*	10	yes	OBL
6. *Xanthium spinosum*	1	____	FACU
7. *Xanthium strumarium*	1	____	FAC
8. *Lolium multiflorum, Picris echioides, Foeniculum vulgare.*	3	____	UPL
50% = _____, 20% = _____	_____	= Total Cover	
Woody Vine Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
% Bare Ground in Herb Stratum _____	% Cover of Biotic Crust _____		

Dominance Test Worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC:	1	(A)
Total Number of Dominant Species Across All Strata:	1	(B)
Percent of Dominant Species That Are OBL, FACW, or FAC:	100	(A/B)

Prevalence Index worksheet:

	Total % Cover of :	Multiply by:
OBL species	11	x1 = 11
FACW species	0	x2 = 0
FAC species	7	x3 = 21
FACU species	6	x4 = 24
UPL species	3	x5 = 15
Column Totals:	27 (A)	71 (B)

Prevalence Index = B/A = 2.63

Hydrophytic Vegetation Indicators:

- ☒ Dominance Test is >50%
- ☒ Prevalence Index is ≤3.0[1]
- ☐ Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ☐ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? **Yes ☒ No ☐**

Remarks: 83% of cover is open water due to recent rain.

Project Site: Parkway Commerce Center Property

SOIL

Sampling Point: 1B

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (Moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-12	2.5 YR 3/1		7.5 YR 4/6				Clay loam	Mottle Abundance/Contrast is CD

[1]Type: C= Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR C**)
- ☐ 1 cm Muck (A9) (**LRR D**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☒ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☐ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☒ Redox Depressions (F8)
- ☐ Vernal Pools (F9)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) **(LRR C)**
- ☐ 2 cm Muck (A10) **(LRR B)**
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ____

Depth (Inches): "

Hydric Soils Present? **Yes** ☒ **No** ☐

Remarks:

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1) **(Nonriverine)**
- ☐ Sediment Deposits (B2) **(Nonriverine)**
- ☐ Drift Deposits (B3) **(Nonriverine)**
- ☐ Surface Soil Cracks (B6)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Biotic Crust (B12)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☒ Oxidized Rhizospheres along Living Roots (C3)
- ☐ Presence of Reduced Iron (C4)
- ☐ Recent Iron Reduction in Tilled Soils (C6)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (2 or more required)

- ☐ Water Marks (B1) **(Riverine)**
- ☒ Sediment Deposits (B2) **(Riverine)**
- ☐ Drift Deposits (B3) **(Riverine)**
- ☐ Drainage Patterns (B10)
- ☐ Dry-Season Water Table (C2)
- ☐ Crayfish Burrows (C8)
- ☒ Saturation Visible on Aerial Imagery (C9)
- ☐ Shallow Aquitard (D3)
- ☐ FAC-Neutral Test (D5)

Field Observations:

Surface Water Present?	Yes	☒	No	☐	Depth (inches): ____
Water Table Present?	Yes	☒	No	☐	Depth (inches): ____
Saturation Present? (includes capillary fringe)	Yes	☒	No	☐	Depth (inches): ____

Wetland Hydrology Present? **Yes** ☒ **No** ☐

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: A distinct topographic depression forms a swale that drains to the rail road tracks. Water sits at the base of the rail road track levee. Primary indicators include heavy algal matting on the vegetation and sediment deposits on the soil surface. Corps hydrology criteria are met at this location.

WETLAND DETERMINATION DATA FORM – Arid West Region

Project Site: Parkway Commerce Center Property City/County: Richmond/Conta Costa Sampling Date: 12/28/12

Applicant/Owner: Panattoni Development Company, LLC State: CA Sampling Point: 2A

Investigator(s): Olberding Environmental Section, Township, Range: _____

Landform (hillslope, terrace, etc.): _____ Local relief (concave, convex, none): ____ Slope (%): 2

Subregion (LRR): _____ Lat: 37^ 58' 40.58" N Long: 122^ 21' 25.74" W Datum: _____

Soil Map Unit Name: Clear Lake Clay NWI classification: _____

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ☒ No ☐ (If no, explain in Remarks.)

Are Vegetation ☐, Soil ☐, or Hydrology ☐ significantly disturbed? Are "Normal Circumstances" present? Yes ☒ No ☐

Are Vegetation ☐, Soil ☐, or Hydrology ☐ naturally problematic? (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ☐ No ☒		
Hydric Soil Present?	Yes ☐ No ☒	**Is the Sampled Area within a Wetland?**	**Yes ☐ No ☒**
Wetland Hydrology Present?	Yes ☐ No ☒		

Remarks: **Data point fails to meet all three Corps wetland criteria. Upland position for comparison to wetland position.**

VEGETATION – Use scientific names of plants.

Tree Stratum (Plot size:_____)	Absolute % Cover	Dominant Species?	Indicator Status
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Sapling/Shrub Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
5. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Herb Stratum (Plot size:_____)			
1. *Picris echioides*	20	yes	UPL
2. *Poaceae*	35	yes	UPL
3. *Rumex crispus*	30	yes	FAC
4. *Lolium multiflorum*	5	____	UPL
5. *Elymus triticoides and Malva parviflora.*	2	____	UPL
6. *Brassica sp*	1	____	FACU
7. *Lotus corniclatus*	2	____	FAC
8. *Geranium sp.*	5	____	FAC
50% = _____, 20% = _____	100	= Total Cover	
Woody Vine Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
% Bare Ground in Herb Stratum _____	% Cover of Biotic Crust _____		

Dominance Test Worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC:	1	(A)
Total Number of Dominant Species Across All Strata:	3	(B)
Percent of Dominant Species That Are OBL, FACW, or FAC:	**33**	(A/B)

Prevalence Index worksheet:

	Total % Cover of :	Multiply by:
OBL species	0	x1 = 0
FACW species	0	x2 = 0
FAC species	37	x3 = 111
FACU species	1	x4 = 4
UPL species	62	x5 = 310
Column Totals:	100 (A)	425 (B)

Prevalence Index = B/A = 4.25

Hydrophytic Vegetation Indicators:

- ☐ Dominance Test is >50%
- ☐ Prevalence Index is ≤3.0[1]
- ☐ Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ☐ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? **Yes ☐ No ☒**

Remarks:

Project Site: Parkway Commerce Center Property

SOIL

Sampling Point: 2A

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (Moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-12	10 YR 3/2		None				Loam	

[1]Type: C= Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR C**)
- ☐ 1 cm Muck (A9) (**LRR D**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☐ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☐ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☐ Redox Depressions (F8)
- ☐ Vernal Pools (F9)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) **(LRR C)**
- ☐ 2 cm Muck (A10) **(LRR B)**
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ____

Depth (Inches): "

Hydric Soils Present? **Yes** ☐ **No** ☒

Remarks:

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1) **(Nonriverine)**
- ☐ Sediment Deposits (B2) **(Nonriverine)**
- ☐ Drift Deposits (B3) **(Nonriverine)**
- ☐ Surface Soil Cracks (B6)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Biotic Crust (B12)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☐ Oxidized Rhizospheres along Living Roots (C3)
- ☐ Presence of Reduced Iron (C4)
- ☐ Recent Iron Reduction in Tilled Soils (C6)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (2 or more required)

- ☐ Water Marks (B1) **(Riverine)**
- ☐ Sediment Deposits (B2) **(Riverine)**
- ☐ Drift Deposits (B3) **(Riverine)**
- ☐ Drainage Patterns (B10)
- ☐ Dry-Season Water Table (C2)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Shallow Aquitard (D3)
- ☐ FAC-Neutral Test (D5)

Field Observations:

Surface Water Present?	Yes	☐	No	☒	Depth (inches): ____
Water Table Present?	Yes	☐	No	☒	Depth (inches): ____
Saturation Present? (includes capillary fringe)	Yes	☐	No	☒	Depth (inches): ____

Wetland Hydrology Present? **Yes** ☐ **No** ☒

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Upland position for comparison to wetland position. No hydrology indicators were observed. Data point located several feet up slope from wetland.

WETLAND DETERMINATION DATA FORM – Arid West Region

Project Site: Parkway Commerce Center Property City/County: Richmond/Conta Costa Sampling Date: 12/28/12

Applicant/Owner: Panattoni Development Company, LLC State: CA Sampling Point: 2B

Investigator(s): Olberding Environmental Section, Township, Range: _____

Landform (hillslope, terrace, etc.): _____ Local relief (concave, convex, none): ____ Slope (%): 2

Subregion (LRR): _____ Lat: 37^ 58' 40.58" N Long: 122^ 21' 25.74" W Datum: _____

Soil Map Unit Name: Clear Lake Clay NWI classification: _____

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ☒ No ☐ (If no, explain in Remarks.)

Are Vegetation ☐, Soil ☐, or Hydrology ☐ significantly disturbed? Are "Normal Circumstances" present? Yes ☒ No ☐

Are Vegetation ☐, Soil ☐, or Hydrology ☐ naturally problematic? (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ☒ No ☐		
Hydric Soil Present?	Yes ☒ No ☐	**Is the Sampled Area within a Wetland?**	**Yes ☒ No ☐**
Wetland Hydrology Present?	Yes ☒ No ☐		

Remarks: **The data point is positioned in a wetland. The three Corps criteria for wetlands are met.**

VEGETATION – Use scientific names of plants.

Tree Stratum (Plot size:_____)	Absolute % Cover	Dominant Species?	Indicator Status
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Sapling/Shrub Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
5. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Herb Stratum (Plot size:_____)			
1. *Rumex crispus*	1	____	FAC
2. *Medicago sp.*	5	____	FACU
3. *Geranium sp.*	5	____	FAC
4. *Cyperus sp.*	1	____	OBL
5. *Lythrum hyssopifolia*	10	yes	OBL
6. *Xanthium spinosum*	1	____	FACU
7. *Xanthium strumarium*	1	____	FAC
8. *Lolium multiflorum, Picris echioides, Foeniculum vulgare.*	3	____	UPL
50% = _____, 20% = _____	_____	= Total Cover	
Woody Vine Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
% Bare Ground in Herb Stratum _____	% Cover of Biotic Crust _____		

Dominance Test Worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC:	1	(A)
Total Number of Dominant Species Across All Strata:	1	(B)
Percent of Dominant Species That Are OBL, FACW, or FAC:	100	(A/B)

Prevalence Index worksheet:

	Total % Cover of :	Multiply by:
OBL species	11	x1 = 11
FACW species	0	x2 = 0
FAC species	7	x3 = 21
FACU species	6	x4 = 24
UPL species	3	x5 = 15
Column Totals:	27 (A)	71 (B)

Prevalence Index = B/A = 2.63

Hydrophytic Vegetation Indicators:

- ☒ Dominance Test is >50%
- ☒ Prevalence Index is ≤ 3.0[1]
- ☐ Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ☐ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? **Yes ☒ No ☐**

Remarks: 83% of cover is open water due to recent rain.

Project Site: Parkway Commerce Center Property

SOIL

Sampling Point: 2B

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (Moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-12	10 YR 3/1		7.5 YR 4/6				Clay loam	Mottle Abundance/Contrast is FF

[1]Type: C= Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR C**)
- ☐ 1 cm Muck (A9) (**LRR D**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☒ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☐ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☒ Redox Depressions (F8)
- ☐ Vernal Pools (F9)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) **(LRR C)**
- ☐ 2 cm Muck (A10) **(LRR B)**
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ____

Depth (Inches): "

Hydric Soils Present? **Yes** ☒ **No** ☐

Remarks:

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1) **(Nonriverine)**
- ☐ Sediment Deposits (B2) **(Nonriverine)**
- ☐ Drift Deposits (B3) **(Nonriverine)**
- ☐ Surface Soil Cracks (B6)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Biotic Crust (B12)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☒ Oxidized Rhizospheres along Living Roots (C3)
- ☐ Presence of Reduced Iron (C4)
- ☐ Recent Iron Reduction in Tilled Soils (C6)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (2 or more required)

- ☐ Water Marks (B1) **(Riverine)**
- ☒ Sediment Deposits (B2) **(Riverine)**
- ☒ Drift Deposits (B3) **(Riverine)**
- ☐ Drainage Patterns (B10)
- ☐ Dry-Season Water Table (C2)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Shallow Aquitard (D3)
- ☐ FAC-Neutral Test (D5)

Field Observations:

Surface Water Present?	Yes	☒	No	☐	Depth (inches): ____
Water Table Present?	Yes	☒	No	☐	Depth (inches): ____
Saturation Present? (includes capillary fringe)	Yes	☒	No	☐	Depth (inches): ____

Wetland Hydrology Present? **Yes** ☒ **No** ☐

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: A distinct topographic depression forms a swale that drains to the rail road tracks. Primary indicators include heavy algal matting on the vegetation and sediment deposits on the soil surface. Corps hydrology criteria are met at this location.

WETLAND DETERMINATION DATA FORM – Arid West Region

Project Site: Parkway Commerce Center Property City/County: Richmond/Conta Costa Sampling Date: 12/28/12

Applicant/Owner: Panattoni Development Company, LLC State: CA Sampling Point: 3A

Investigator(s): Olberding Environmental Section, Township, Range: _____

Landform (hillslope, terrace, etc.): _____ Local relief (concave, convex, none): ____ Slope (%): 2

Subregion (LRR): _____ Lat: 37^ 58' 40.58" N Long: 122^ 21' 25.74" W Datum: _____

Soil Map Unit Name: Clear Lake Clay NWI classification: _____

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ☒ No ☐ (If no, explain in Remarks.)

Are Vegetation ☐, Soil ☐, or Hydrology ☐ significantly disturbed? Are "Normal Circumstances" present? Yes ☒ No ☐

Are Vegetation ☐, Soil ☐, or Hydrology ☐ naturally problematic? (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ☐	No ☒	
Hydric Soil Present?	Yes ☐	No ☒	**Is the Sampled Area within a Wetland?** **Yes ☐ No ☒**
Wetland Hydrology Present?	Yes ☐	No ☒	

Remarks: **Data point fails to meet all three Corps wetland criteria. Upland position for comparison to channel position.**

VEGETATION – Use scientific names of plants.

Tree Stratum (Plot size:_____)	Absolute % Cover	Dominant Species?	Indicator Status
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Sapling/Shrub Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
5. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Herb Stratum (Plot size:_____)			
1. *Picris echioides*	20	yes	UPL
2. *Poaceae*	35	yes	UPL
3. *Rumex crispus*	30	yes	FAC
4. *Lolium multiflorum*	5	____	UPL
5. *Elymus triticoides and Malva parviflora.*	2	____	UPL
6. *Brassica sp*	1	____	FACU
7. *Lotus corniclatus*	2	____	FAC
8. *Geranium sp.*	5	____	FAC
50% = _____, 20% = _____	100	= Total Cover	
Woody Vine Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
% Bare Ground in Herb Stratum _____	% Cover of Biotic Crust _____		

Dominance Test Worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC:	1	(A)
Total Number of Dominant Species Across All Strata:	3	(B)
Percent of Dominant Species That Are OBL, FACW, or FAC:	**33**	(A/B)

Prevalence Index worksheet:

	Total % Cover of :	Multiply by:
OBL species	0	x1 = 0
FACW species	0	x2 = 0
FAC species	37	x3 = 111
FACU species	1	x4 = 4
UPL species	62	x5 = 310
Column Totals:	100 (A)	425 (B)

Prevalence Index = B/A = 4.25

Hydrophytic Vegetation Indicators:

- ☐ Dominance Test is >50%
- ☐ Prevalence Index is ≤3.0[1]
- ☐ Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ☐ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? **Yes ☐ No ☒**

Remarks:

Project Site: Parkway Commerce Center Property

SOIL

Sampling Point: 3A

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (Moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-12	10 YR 3/2		None				Loam	

[1]Type: C= Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR C**)
- ☐ 1 cm Muck (A9) (**LRR D**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☐ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☐ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☐ Redox Depressions (F8)
- ☐ Vernal Pools (F9)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) **(LRR C)**
- ☐ 2 cm Muck (A10) **(LRR B)**
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ____

Depth (Inches): "

Hydric Soils Present? **Yes** ☐ **No** ☒

Remarks:

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1) **(Nonriverine)**
- ☐ Sediment Deposits (B2) **(Nonriverine)**
- ☐ Drift Deposits (B3) **(Nonriverine)**
- ☐ Surface Soil Cracks (B6)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Biotic Crust (B12)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☐ Oxidized Rhizospheres along Living Roots (C3)
- ☐ Presence of Reduced Iron (C4)
- ☐ Recent Iron Reduction in Tilled Soils (C6)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (2 or more required)

- ☐ Water Marks (B1) **(Riverine)**
- ☐ Sediment Deposits (B2) **(Riverine)**
- ☐ Drift Deposits (B3) **(Riverine)**
- ☐ Drainage Patterns (B10)
- ☐ Dry-Season Water Table (C2)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Shallow Aquitard (D3)
- ☐ FAC-Neutral Test (D5)

Field Observations:

Surface Water Present?	Yes	☐	No	☒	Depth (inches): ____
Water Table Present?	Yes	☐	No	☒	Depth (inches): ____
Saturation Present? (includes capillary fringe)	Yes	☐	No	☒	Depth (inches): ____

Wetland Hydrology Present? **Yes** ☐ **No** ☒

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: Upland position for comparison to wetland position. No hydrology indicators were observed. Data point located several feet up slope from wetland.

WETLAND DETERMINATION DATA FORM – Arid West Region

Project Site: Parkway Commerce Center Property City/County: Richmond/Conta Costa Sampling Date: 12/28/12

Applicant/Owner: Panattoni Development Company, LLC State: CA Sampling Point: 3B

Investigator(s): Olberding Environmental Section, Township, Range: _____

Landform (hillslope, terrace, etc.): _____ Local relief (concave, convex, none): ____ Slope (%): 2

Subregion (LRR): _____ Lat: 37^ 58' 40.58" N Long: 122^ 21' 25.74" W Datum: _____

Soil Map Unit Name: Clear Lake Clay NWI classification: _____

Are climatic / hydrologic conditions on the site typical for this time of year? Yes ☒ No ☐ (If no, explain in Remarks.)

Are Vegetation ☐, Soil ☐, or Hydrology ☐ significantly disturbed? Are "Normal Circumstances" present? Yes ☒ No ☐

Are Vegetation ☐, Soil ☐, or Hydrology ☐ naturally problematic? (If needed, explain any answers in Remarks.)

SUMMARY OF FINDINGS – Attach site map showing sampling point locations, transects, important features, etc.

Hydrophytic Vegetation Present?	Yes ☒	No ☐		
Hydric Soil Present?	Yes ☒	No ☐	**Is the Sampled Area within a Wetland?**	**Yes ☒ No ☐**
Wetland Hydrology Present?	Yes ☒	No ☐		

Remarks: **The data point is positioned in a seasonal wetland. The three Corps criteria for wetlands are met.**

VEGETATION – Use scientific names of plants.

Tree Stratum (Plot size:_____)	Absolute % Cover	Dominant Species?	Indicator Status
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Sapling/Shrub Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
3. _____	_____	____	____
4. _____	_____	____	____
5. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
Herb Stratum (Plot size:_____)			
1. *Rumex crispus*	1	____	FAC
2. *Medicago sp.*	5	____	FACU
3. *Geranium sp.*	5	____	FAC
4. *Cyperus sp.*	1	____	OBL
5. *Lythrum hyssopifolia*	10	yes	OBL
6. *Xanthium spinosum*	1	____	FACU
7. *Xanthium strumarium*	1	____	FAC
8. *Lolium multiflorum, Picris echioides, Foeniculum vulgare.*	3	____	UPL
50% = _____, 20% = _____	_____	= Total Cover	
Woody Vine Stratum (Plot size:_____)			
1. _____	_____	____	____
2. _____	_____	____	____
50% = _____, 20% = _____	_____	= Total Cover	
% Bare Ground in Herb Stratum _____	% Cover of Biotic Crust _____		

Dominance Test Worksheet:

Number of Dominant Species That Are OBL, FACW, or FAC:	1	(A)
Total Number of Dominant Species Across All Strata:	1	(B)
Percent of Dominant Species That Are OBL, FACW, or FAC:	100	(A/B)

Prevalence Index worksheet:

	Total % Cover of :	Multiply by:	
OBL species	11	x1 =	11
FACW species	0	x2 =	0
FAC species	7	x3 =	21
FACU species	6	x4 =	24
UPL species	3	x5 =	15
Column Totals:	27 (A)		71 (B)

Prevalence Index = B/A = 2.63

Hydrophytic Vegetation Indicators:

- ☒ Dominance Test is >50%
- ☒ Prevalence Index is ≤3.0[1]
- ☐ Morphological Adaptations[1] (Provide supporting data in Remarks or on a separate sheet)
- ☐ Problematic Hydrophytic Vegetation[1] (Explain)

[1]Indicators of hydric soil and wetland hydrology must be present, unless disturbed or problematic.

Hydrophytic Vegetation Present? **Yes ☒ No ☐**

Remarks: 83% of cover is open water due to recent rain.

Project Site: Parkway Commerce Center Property

SOIL

Sampling Point: 3B

Profile Description: (Describe to the depth needed to document the indicator or confirm the absence of indicators.)

Depth	Matrix		Redox Features					
(inches)	Color (moist)	%	Color (Moist)	%	Type[1]	Loc[2]	Texture	Remarks
0-12	2.5 YR 2/1		7.5 YR 4/6				Clay loam	Mottle Abundance/Contrast is FD

[1]Type: C= Concentration, D=Depletion, RM=Reduced Matrix, CS=Covered or Coated Sand Grains. [2]Location: PL=Pore Lining, M=Matrix.

Hydric Soil Indicators: (Applicable to all LRRs, unless otherwise noted.)

- ☐ Histosol (A1)
- ☐ Histic Epipedon (A2)
- ☐ Black Histic (A3)
- ☐ Hydrogen Sulfide (A4)
- ☐ Stratified Layers (A5) (**LRR C**)
- ☐ 1 cm Muck (A9) (**LRR D**)
- ☐ Depleted Below Dark Surface (A11)
- ☐ Thick Dark Surface (A12)
- ☐ Sandy Mucky Mineral (S1)
- ☐ Sandy Gleyed Matrix (S4)
- ☐ Sandy Redox (S5)
- ☐ Stripped Matrix (S6)
- ☐ Loamy Mucky Mineral (F1)
- ☒ Loamy Gleyed Matrix (F2)
- ☐ Depleted Matrix (F3)
- ☐ Redox Dark Surface (F6)
- ☐ Depleted Dark Surface (F7)
- ☒ Redox Depressions (F8)
- ☐ Vernal Pools (F9)

Indicators for Problematic Hydric Soils[3]:

- ☐ 1 cm Muck (A9) **(LRR C)**
- ☐ 2 cm Muck (A10) **(LRR B)**
- ☐ Reduced Vertic (F18)
- ☐ Red Parent Material (TF2)
- ☐ Other (Explain in Remarks)

[3]Indicators of hydrophytic vegetation and wetland hydrology must be present, unless disturbed or problematic.

Restrictive Layer (if present):

Type: ____

Depth (Inches): "

Hydric Soils Present? **Yes** ☒ **No** ☐

Remarks:

HYDROLOGY

Wetland Hydrology Indicators:

Primary Indicators (minimum of one required; check all that apply)

- ☐ Surface Water (A1)
- ☐ High Water Table (A2)
- ☐ Saturation (A3)
- ☐ Water Marks (B1) **(Nonriverine)**
- ☐ Sediment Deposits (B2) **(Nonriverine)**
- ☐ Drift Deposits (B3) **(Nonriverine)**
- ☐ Surface Soil Cracks (B6)
- ☐ Inundation Visible on Aerial Imagery (B7)
- ☐ Water-Stained Leaves (B9)
- ☐ Salt Crust (B11)
- ☐ Biotic Crust (B12)
- ☐ Aquatic Invertebrates (B13)
- ☐ Hydrogen Sulfide Odor (C1)
- ☒ Oxidized Rhizospheres along Living Roots (C3)
- ☐ Presence of Reduced Iron (C4)
- ☐ Recent Iron Reduction in Tilled Soils (C6)
- ☐ Thin Muck Surface (C7)
- ☐ Other (Explain in Remarks)

Secondary Indicators (2 or more required)

- ☐ Water Marks (B1) **(Riverine)**
- ☒ Sediment Deposits (B2) **(Riverine)**
- ☒ Drift Deposits (B3) **(Riverine)**
- ☐ Drainage Patterns (B10)
- ☐ Dry-Season Water Table (C2)
- ☐ Crayfish Burrows (C8)
- ☐ Saturation Visible on Aerial Imagery (C9)
- ☐ Shallow Aquitard (D3)
- ☐ FAC-Neutral Test (D5)

Field Observations:

Surface Water Present?	Yes	☒	No	☐	Depth (inches): ____
Water Table Present?	Yes	☒	No	☐	Depth (inches): ____
Saturation Present? (includes capillary fringe)	Yes	☒	No	☐	Depth (inches): ____

Wetland Hydrology Present? **Yes** ☒ **No** ☐

Describe Recorded Data (stream gauge, monitoring well, aerial photos, previous inspections), if available:

Remarks: A distinct topographic depression forms a swale that drains to the rail road tracks. Primary indicators include heavy algal matting on the vegetation and sediment deposits on the soil surface, and heavy presence of seed shrimp. Corps hydrology criteria are met at this location.

Draft Environmental Impact Report
Upper York Creek
Ecosystem Restoration Project

Napa County, California

(State Clearinghouse No. 2006092096)

CEQA Lead Agency

City of St. Helena
1480 Main Street
St. Helena, CA 95474

Contact: Jonathon Goldman
Director of Public Works/City Engineer
707-968-2658

A Joint Project of the

City of St. Helena, Napa County, California
U.S. Army Corps of Engineers, San Francisco District, South Pacific Division

April 20, 2007

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Executive Summary

The proposed action is the *Upper York Creek Ecosystem Restoration Project* at the Upper York Creek Dam and Reservoir (UYCD), also referred to as the Upper St. Helena Dam. The project sponsors, the City of St. Helena (City) and the U.S. Army Corps of Engineers, San Francisco District, South Pacific Division (Corps), propose to remove or modify the dam and appurtenances, remove accumulated sediment, and restore the local ecology of York Creek. The City is serving as Lead Agency under the California Environmental Quality Act (CEQA) and has prepared this Draft Environmental Impact Report (DEIR). The purpose of the DEIR is to inform public agency decision-makers and the general public of the significant environmental effects of the project, to identify ways to minimize the significant effects, and to describe reasonable alternatives to the project (CCR §15121).

Upper St. Helena Dam is located approximately 1.5 miles upstream of the City of St. Helena on Spring Mountain Road (Figure 1). Constructed in 1900 to expand water capacity for the rapidly growing City of St. Helena and the local wine industry, UYCD was designed with a storage capacity of 10 million gallons. The earthen dam was originally 35 feet high and 140 feet long and was composed of more than 12,000 cubic yards of channel-bed material that, when excavated, formed the upper reservoir. The dam was modified in the 1930s with the installation of a concrete spillway along Spring Mountain Road and a drop-inlet extension to the out-flow culvert. The dam height was raised 15 feet, and the sluice gates were removed. With these alterations, the dam can no longer be operated as a managed water-storage facility, nor is it large enough to provide downstream flood control.

The reservoir traps the annual supply of gravel and fine sediments produced by the watershed above UYCD. Sediment accumulates in the reservoir at a rate of 1,000 to 5,000 cubic yards per year depending on the magnitude of winter storm events. With total sediment storage potential of 28,000 cubic yards of material, the reservoir pool area is filled every 15-20 years. Over 100 years of sediment trapping has altered the geomorphology of York Creek downstream of UYCD and is thought to be a significant contributing factor to the six to eight feet of bed incision that has occurred in the Napa River over the last century (Stillwater Sciences 2002). Regular maintenance of the dam is required, including annual debris removal from the drop-inlet and spillway to maintain storm water passage and periodic removal of accumulated sediment.

Four documented incidents of accidental, catastrophic releases of fine sediment have occurred during maintenance activities at UYCD. These low-flow sediment discharges have resulted in mortality of fish and other aquatic organisms over the entire stretch of York Creek below the dam. York Creek contains the highest quality steelhead spawning and rearing habitat in the central Napa River Basin, and maintenance of this habitat is very important to support overall Napa River steelhead populations (NCRCD 2005). The recurring fish kills, together with the dam being a total barrier to aquatic migration, initiated formal enforcement actions from the California Department of Fish and Game (CDFG) and NOAA Fisheries Service requiring removal of UYCD.

UYCD significantly impacts critical biological resources in the York Creek and Napa River watersheds. The objectives for removing or modifying UYCD are to restore fish passage for steelhead *(Oncorhynchus mykiss*), reduce potential for future downstream low-flow fine

sediment releases and their associated fish kills, and allow for the restoration of approximately 3 acres of degraded riparian and aquatic habitat surrounding the dam and reservoir. Maintaining the stability of the adjoining Spring Mountain Road is considered a project constraint that must be addressed adequately to achieve project success (Corps 2006a).

The City first applied to the Corps Regulatory Branch for permission to address the dam in 1993. This application and a revised application in 1994 were determined to be incomplete. In 2000, the City Council resolved that the Upper St. Helena Dam should at least be breached to allow fish passage. The City formed a partnership with the Corps Civil Works Branch in 2001 that initiated planning for implementation of the *Upper York Creek Ecosystem Restoration Project.* As part of the larger fish-passage restoration project, removal of a City water-diversion structure 0.5 miles downstream of the dam was completed in 2004.

The Corps initially evaluated seven preliminary alternatives in the feasibility study for the *Upper York Creek Ecosystem Restoration Project*. Three were eliminated early in the analysis as ineffective or inefficient. Both a small and a large notch in the dam were evaluated, but it was determined that geotechnical slope stability constraints outweighed the small gain in floodplain area offered by the large notch alternative. (Corps 2006b)

Based upon the Corps' recommendations, no project and three action alternatives are evaluated herein. The alternatives analyzed are discussed briefly here and in greater detail in Section 2:

- No Project – The historic Upper St. Helena Dam would not be altered from its current configuration. No ecosystem restoration measures would be implemented. Regular maintenance dredging of the reservoir would be required to offset the continued deposition of materials behind the dam to prevent sediment releases.
- Preferred Alternative – Small Notch – A small notch would be cut into the dam to allow for fish passage and habitat connectivity above and below the dam. A 23-foot wide stream channel would be restored. The existing spillway adjacent to Spring Mountain Road would remain intact and be filled with sediment from the project area, and two layers of inclined screw anchors and subdrains would be installed to provide slope stability.
- Full Dam Removal – The entire dam, right wall of the spillway (looking downstream), and the drop inlet would be removed. A 23-foot wide restored stream channel with a 30-foot wide floodplain bench would be created through the dam area. Three layers of inclined screw anchors and subdrains would provide slope stability for Spring Mountain Road.
- Fish Ladder – This alternative would create a concrete step-pool/weir fish ladder cut into the dam. Fish passage and habitat connectivity would be partially restored. The Fish Ladder Alternative does not allow for natural sediment transport and would require on-going maintenance to prevent it from becoming clogged with sediment and other debris.

A cross sectional rendering and plan views of the three action alternatives are provided in Figures 2-5 in Section 2. For cross-referencing with Corps documents, these alternatives are identified by the Corps as No Project, Alternative 1 (Full Dam Removal), Alternative 2B (Small

Notch), and Alternative 3 (Fish Ladder). The alternatives are identified by name only in this document; no numbering is used.

Sediment removed from the dam and reservoir will be placed at one or more locations in or near the City of St. Helena, including a private upland vineyard (Spring Mountain Vineyard), Lower York Creek Reservoir, and Clover Flat landfill. Early in the planning process, the City conducted a preliminary investigation to evaluate if material from the restoration project could be useful to address flood control issues at Fulton Lane within the City limits. However, neither the form nor the location of the flood control work has been established, so impacts cannot be assessed at this time. If the City and private landowners reach an agreement on how to address flooding at Fulton Lane, additional environmental review will occur.

The Initial Study (Appendix 1 [PCI 2006]) determined that further analyses of potential impacts to mineral resources, public services, utilities, recreation, land use/planning, and population/housing were not required. With removal of Fulton Lane as a potential reuse site, no impacts to agricultural resources will occur. In fact, beneficial effects will occur at Spring Mountain Vineyard where materials from the reservoir will be used to amend the currently fallow land.

Full discussion of the following resources, potential impacts from each alternative, and proposed mitigation measures is found in Section 3:

- Aesthetics
- Air Quality
- Biological Resources
- Cultural Resources
- Geology, Soils, Landslides, and Seismic Activity
- Global Climate Change
- Hazardous Materials
- Hydrology and Hydraulics
- Noise
- Transportation and Traffic
- Water Quality

Table ES-1 below summarizes impacts of the proposed project's alternatives as analyzed in the DEIR, identifies the level of significance of each impact before mitigation, recommends feasible and appropriate mitigation measures, and identifies the level of significance after mitigation for each impact. The level of significance is included for each impact based on the following classification system: significant unavoidable impact (SU), potentially significant and unavoidable (PSU), less than significant (LS), and not applicable (N/A). Section 1.6 summarizes compliance with applicable laws and regulations.

The City and the Corps have worked with local, state, and federal agencies and involved the public during the planning process. No significant public controversy regarding the proposed action has emerged to date. The public involvement process and timeline are discussed in Section 1.5.

Issues to be Resolved for the Action Alternatives

The City has identified several issues to be resolved involving implementation of all of the action alternatives, including potentially significant impacts to aesthetics and noise from project activities at the Lower York Creek Reservoir, to historic resources through removal of the 107-year-old dam, and on flooding in the lower York Creek watershed. A summary of these issues and mitigation measures with references to detailed discussion in Section 3 follows. An impact analysis is found in Section 4.

- **Potentially significant and unavoidable impact to aesthetics at the Lower York Creek Reservoir.**

  Placement of materials along the shore of the Lower York Creek Reservoir will change the water surface area (*AES-NP4* in Section 3.1). A revegetation and mitigation and monitoring plan has been prepared to restore wetland and riparian habitat at the lower reservoir (Appendix 2, *Lower York Creek Reservoir Revegetation and Mitigation Plan*). Nevertheless, a change in the view of water surface area may be considered significant by neighbors.

- **Potentially significant and unavoidable short-term impact from noise at the dam and disposal sites.**

  Project activities will result in a temporary increase in noise at the project sites (*NOI-NP2* in Section 3.9). Work will only occur during daylight hours, and efforts will be made to efficiently schedule construction activities to limit impacts to neighbors at the Lower York Creek Reservoir. The long-term benefit to the York Creek ecosystem will outweigh short-term impacts from construction.

- **Significant and unavoidable impact to cultural resources through removal of this historic Upper St. Helena Dam and associated structures.**

  The dam is a unique historical resource, significant for its role in the development of the City of St. Helena, and alteration or removal will be a significant impact (*CUL-PA1* in Section 3.4). No feasible alternatives could be identified that meet the project objectives to remove the barrier to fish passage and restore habitat connectivity. Therefore, the impact is considered significant and unavoidable in order to improve the aquatic environment of York Creek for steelhead and other species.

- **Potentially significant and unavoidable increase in the frequency and severity of flooding in lower York Creek due to sediment aggradation.**

  Lower York Creek through the City has existing flood hazards, with limited areas downstream of Highway 29 regularly experiencing overbank flooding. The 2005 New Years Eve storm, a 25-year event, caused extensive flood damage to wineries, vineyards, and the Culinary Institute of America's dormitories. Coarse material that has annually been trapped in the Upper York Creek Reservoir would be delivered to the lower reaches of York Creek (*H&H-PA1* in Section 3.8). This sediment may accumulate over time in areas of reduced channel competence and locally increase flood hazards. To minimize potential impacts of sedimentation on flood capacity, a channel monitoring program by the City shall be implemented that documents changes in channel cross

section area on a regular, pre-determined schedule; tracks sediment accumulation at bridges and other critical locations; provides guidance on acceptable levels of aggradation; and establishes a mitigation plan should channel aggradation reach a point that it significantly impacts channel capacity and flooding.

Issues to be Resolved for the No Project Alternative
Issues to be resolved in the No Project Alternative include significant and unavoidable impacts to aesthetics, biological resources, global climate change, and noise.

- **Potentially significant and unavoidable impact to aesthetics at the Lower York Creek Reservoir.**

  As discussed above in the action alternatives, placement of materials along the shore of the Lower York Creek Reservoir may change the water surface area in a way that is considered significant by neighbors. (See discussion above and in *AES-NP4* in Section 3.1.)

- **Significant and unavoidable impact to biological resources at the Upper York Creek Reservoir.**

  The No Project Alternative would result in two significant and unavoidable impacts to biological resources (*BIO-NP1* and *BIO-NP2* in Section 3.3). The Upper St. Helena Dam would continue to act as an unnatural migration barrier to fish and other aquatic species, and the Upper York Creek Dam and Reservoir would continue to disrupt the natural transport of materials and organisms, reducing their availability for downstream habitat and food and adversely affecting aquatic species and habitat. No mitigation can be provided.

- **Potentially significant and unavoidable cumulative impact to global climate change.**

  On-going actions to remove sediment and debris will increase carbon dioxide by committing the City to repeated maintenance activities (*GCC-NP1* in Section 3.6). The City will seek contractors who use biodiesels and/or other emission-reducing strategies.

- **Potentially significant and unavoidable impact from noise at the dam and disposal sites.**

  As with the action alternatives, noise levels will increase due to maintenance activities. (See discussion above and in *NOI-NP2* in Section 3.9.)

Additional Issue to be Resolved for the Fish Ladder Alternative
In addition to the impacts to aesthetics and noise discussed for the action alternatives above, the Fish Ladder Alternative would result in a significant and unavoidable impact to biological resources. It would also involve a potentially significant cumulative impact to global climate change identified in the No Project Alternative discussion above (*GCC-NP1* in Section 3.6).

- **Significant, long-term, and unavoidable impact to biological resources.**

  The fish ladder would act as a partial, unnatural, upstream and downstream migration barrier to fish and other aquatic species (*BIO-FL1* in Section 3.3). The Corps estimates it

will provide between 65% and 95% effectiveness for fish passage (Corps 2006a). No mitigation, other than on-going maintenance, is proposed.

Of the alternatives analyzed, the Small Notch is considered environmentally superior because it provides for the highest levels of ecosystem restoration and protection, while providing for greater slope stability than the Full Dam Removal Alternative. Beneficial environmental effects of the Preferred Small Notch Alternative include restored access to 1.7 miles of high quality habitat for steelhead trout; restored habitat connectivity and unimpeded migration for aquatic and terrestrial organisms; elimination of the threat of harmful low-flow fine sediment releases; and reestablished natural sediment transport processes to improve in-stream habitat in York Creek and the Napa River. Long-term impacts associated with the Preferred Small Notch Alternative include removal of an historic structure (Upper St. Helena Dam) and a possibility for changes in existing flood heights due to localized, temporary channel aggradation in lower York Creek. Construction activities are likely to produce short-term significant impacts to noise levels, traffic congestion, and aesthetics at the Upper and Lower York Creek Reservoirs.

1 Proposed Action

1.1 Project Description and Objectives

The *Upper York Creek Ecosystem Restoration Project* is jointly sponsored by the City of St. Helena (City) and the U.S. Army Corps of Engineers (Corps). The project involves alteration or removal of the Upper St. Helena Dam and restoration of the former reservoir area into a natural creek channel and native riparian corridor. Project objectives include:

- **Improved fish passage and habitat connectivity.** The project will provide upstream passage to 1.7 miles of spawning and rearing habitat for steelhead *(Oncorhynchus mykiss)* and habitat connectivity for both anadromous and resident fish and other aquatic and riparian species.

- **Reduced future downstream habitat degradation and fish kills.** The project will prevent potentially detrimental sediment releases during summer low-flow conditions that have caused fish and aquatic organism kills in the past. It will provide a permanent solution to prevent short-term aquatic habitat impairment associated with possible dam breach/failure.

- **Habitat restoration.** The project will restore approximately 3 acres of degraded riparian and aquatic habitat within the existing upper dam and reservoir area. Restoration of natural sediment transport processes will occur through the project site, with potential habitat improvement occurring in downstream reaches of York Creek and the Napa River. Disturbed areas in the project site will be revegetated with a palette of multistory native plants. A diversity of habitats for aquatic, riparian, and terrestrial animals will be provided.

1.2 Project Location

The proposed project is located on the western edge of the Napa Valley near the City of St. Helena. The Napa River runs north to south on the east side of the City, and several of its tributaries flow through the City eastward from the hills of the Mayacmas Mountains. The primary project site is on York Creek, which is one of these tributaries. The proposed project will take place at the Upper York Creek Dam and Reservoir (Upper St. Helena Dam), which is located at 38º 30′ 48″ N, 122º 30′ 9″ W, in the SW ¼, Section 26, Range 6 West, Township 8 North, Mt. Diablo Base and Meridian of the USGS St. Helena Quadrangle, St. Helena, Napa County, California (Figure 1). The project area is zoned AW (Agriculture, Watershed or Open Space Lands).

The project area also includes 3 ancillary sites that are under consideration for reuse or disposal of materials removed during project activities: an adjacent private vineyard, Lower York Creek Reservoir (LYCR or lower reservoir), and Clover Flat landfill. Spring Mountain Vineyard (SMV) is accessed from 2805 Spring Mountain Road, which is located approximately 0.5 miles downhill and south of the upper reservoir; there is also an access road approximately 0.5 miles uphill. The lower reservoir is located less than 0.5 miles downhill on the east side of Spring Mountain Road. Clover Flat is located at 4380 Silverado Trail near Calistoga, 8.5 miles north of the main project site.

1.3 Characteristics of the Project Area

York Creek runs approximately 7.24 miles from its headwaters to its confluence with the Napa River (Corps 2006b). It descends steeply over the first portion of its range then flattens out as it crosses the floodplain of the Napa River. It drains a watershed of approximately 4.4 square miles (Watershed Information and Conservancy Center of Napa County). At the site of the upper reservoir, York Creek traverses a steep valley cut from serpentinite and sheared shale. Vegetation is a mixture of redwood forest, mixed evergreen forest, riparian woodland, foothill pine-oak woodland, freshwater wetland, non-native grassland, vineyards, and ruderal areas.

According to the Napa County Resource Conservation District (NCRCD), York Creek is one of the most significant spawning and rearing streams in the Napa River watershed for Central California Coast ESU steelhead, which are listed as threatened under the federal Endangered Species Act. It has been designated as critical habitat for steelhead by NOAA Fisheries Service. The Napa River watershed is considered one of the key anadromous fish streams within the San Francisco Bay (NCRCD 2005).

The Upper St. Helena Dam is a total barrier to fish passage. It is comprised of an earthen structure, 140 feet wide and 50 feet high. The outlet is a standpipe that collects water near the downstream edge of the reservoir and discharges it through a stone culvert in the bottom center of the dam. The reservoir behind the dam is full of sediment (approximately 28,000 cubic yards, essentially eliminating any water storage capacity) and is no longer used to provide water for the City's needs. The dam itself is made up of approximately 12,000 cubic yards of material. (Corp 2006a)

There are two spillways: the original on the south and a newer, functioning spillway between the reservoir and Spring Mountain Road that was built in 1933. The left wall of the newer spillway (looking downstream) provides structural support to the road, which connects St. Helena to Santa Rosa in Sonoma County.

The reuse site at Spring Mountain Vineyard is a fallow field adjacent to the upper reservoir that will be replanted with grape vines. Lower York Creek Reservoir currently has a storage capacity of approximately 156 acre-feet; it provides untreated water for a portion of the City's agricultural irrigation and construction water demands. The approved disposal facility being considered is Clover Flat landfill, which is located approximately 8.5 miles from the upper reservoir.

1.4 Project Background and History

York Creek was first dammed in 1871. The current dam was built upstream of the original reservoir in 1900 to extend the City's water delivery system to match its expanding development (Eastman 2003). In 1933, a new concrete overflow spillway was added along the eastern edge of the reservoir and dam immediately adjacent to Spring Mountain Road, the crest of the dam was raised 15 feet to its current height of 50 feet, the standpipe outlet was installed, and other incidental modifications were made.

On July 28, 1992, during routine maintenance of the reservoir outlet, there was an accidental sediment discharge downstream of the dam. This significant release resulted in a silt discharge within the streambed from the face of the dam to York Creek's confluence with the Napa River.

The depth of the silt varied from heavy deposits (up to 18 inches) just below the dam and continuing downstream for about 0.5 miles, gradually thinning until only a light covering of fine silt was deposited at the confluence with the Napa River. According to a CDFG letter dated July 30, 1992, this sediment release was the fourth since 1965. In each incidence, "dense anaerobic sediments, high in toxic hydrogen sulfide, were released from the dam and deposited in pools and riffle areas downstream, quickly suffocating and burying all fish and aquatic invertebrates within a mile or more of the dam."

After this discharge, CDFG filed a complaint with the Napa County District Attorney. In September of 1992, CDFG and the City concluded that the City should remove the existing earthen dam structure on York Creek. An agreement was signed obligating the City to remove the dam, stabilize silt, remove silt that had filtered downstream, and take certain precautions to preserve the stability and natural character of the area.

In October 1993, the City applied to the Corps Regulatory Branch for a federal Clean Water Act Section 404 permit to remove the dam. This application and a revised application in 1994 were determined to be incomplete. At the time of the initial 1993-1994 coordination with the Corps, steelhead in the central California coast Evolutionarily Significant Unit (CCC ESU) were not yet listed pursuant to the federal ESA, and the Corps was not obligated to initiate consultation with NOAA Fisheries Service.

On August 18, 1997, NOAA Fisheries Service listed CCC ESU steelhead as threatened with loss of habitat and threats to their current range. In August of 1998, the City sent a letter to Corps Regulatory Branch requesting that they reactivate the previous permit application for dam removal. This request was declined, citing lack of adequate information to evaluate impacts to the aquatic environment from the project.

In October 2000, a letter was sent from NOAA Fisheries Service law enforcement to the City Manager that provided clarification about the City's potential liabilities under the ESA if the Upper St. Helena Dam were to remain in place. On November 21, 2000, the City Attorney sent NOAA Fisheries Service, Corps Regulatory Branch, CDFG, and the Napa County District Attorney a letter explaining that it was the City Council's position that the Upper St. Helena Dam should at least be breached to allow fish passage and that "the downstream diversion structure should be modified so that it is not a barrier or impediment to the passage of steelhead [5]."

Representatives from NOAA Fisheries Service, the City, Department of Water Resources (DWR), CDFG, Corps Regulatory, Corps Civil Works, and the Napa County District Attorney's Office attended a meeting in early 2001 to discuss the project. DWR representatives stated their intention to assist the City by providing planning and permitting services for the project to modify the diversion structure and to remove the Upper St. Helena Dam. Because in-stream work and dewatering were necessary to correct the adverse effects on steelhead, NOAA Fisheries Service advised that the dam removal or modification project would require formal consultation under Section 7 of the federal ESA.

[5] The City removed the lower diversion in 2004. One-half mile of aquatic habitat was opened to the base of the Upper St. Helena Dam. The modifications involved removal of the concrete structure, creation of cascading steps with resting pools, bank stabilization, and revegetation with native species.

downstream migration of smolts, as well as local migration and dispersal of other aquatic species. In addition, the fish ladder is expected to clog with sediment and debris 4 to 7 times each winter, thereby further restricting steelhead passage (Corps 2006a).

The Fish Ladder Alternative allows for the removal of 37% of the sediment behind the dam and removal of 52% of the actual dam structure. The alternative offers limited sediment transport, which paradoxically improves when the ladder is allowed to fill up with debris. The Fish Ladder Alternative would have little impact on the existing support structures for Spring Mountain Road, and, therefore, no additional geotechnical stabilization measures would be required (Section 3.5, Geology, Soils, Landslides, and Seismic Activity).

Construction of the fish ladder would generate approximately 19,000 cubic yards of materials: 1,000 from downstream of the dam, 8,000 cubic yards of dam material, and 10,000 yards of sediment (Corps 2006a).

Upstream of the ladder, a stream channel would be reconstructed on top of approximately 12 feet of sediment with a gradient of 3% instead of the 5% gradient in both the Small Notch and Full Removal Alternatives. Slopes created by sediment removal would be planted with native trees and shrubs. The Fish Ladder Alternative would include 0.4 acres of bank zone, 0.9 acres of terrace zone, and 0.6 acres of riparian influenced slope for a total of 1.9 acres of restored habitat acreage (Corps 2006a). Not including the fish ladder itself, slightly less than 1.5 acres of steelhead spawning and rearing habitat would be opened.

2.2.5 Summary of Alternatives

Table 2. Comparison of Objectives Met in Relation to Each Alternative.

Alternative	Objectives				Constraint
	Improve aquatic passage	Provide natural sediment transport	Reduce future downstream fish kills and habitat degradation	Habitat restoration	Slope stability
No Project	No	No	No	No	No change
Small Notch	Yes. Equivalent to natural up- and downstream conditions	Yes	Eliminates threat of sediment spill	2.0 acres of restored riparian habitat and 1.7 miles of restored creek in historic channel	Requires reinforcement measures
Full Removal	Yes. Equivalent to natural up- and downstream conditions	Yes	Eliminates threat of sediment spill	2.2 acres of restored riparian habitat and 1.7 miles of restored creek in historic channel	Requires highest level of reinforcement measures

called rainbow trout. Unlike steelhead, rainbow trout are not protected under the federal Endangered Species Act.

Environmental requirements for steelhead vary by season and life stage. Optimal water temperatures for steelhead range from 10 to 15°C, with an upper lethal limit of 20°C. Rearing salmonids require a high level of dissolved oxygen, at least 80%, with a minimum temporary reduction no lower than 5.0 mg/l. Deposited and suspended sediment plays a significant role in their ability to successfully spawn and rear. Optimal upstream migration water velocities range from 40 to 90 cm/s and a minimum stream depth of 13 cm.

Population Estimates

> "Napa River steelhead populations have been greatly reduced from historical levels. It is estimated that the Napa River watershed supported a population of approximately 8,000 adult steelhead as recently as 100 years ago. The current steelhead population is unknown due to a lack of quantitative data. Recent basin wide surveys estimate the population to be between 200 and 1,000 adult steelhead (Stillwater Sciences, 2002; EcoTrust, 2001). NOAA Fisheries listed steelhead as a threatened species in Napa County in August 1997. Spawning adult steelhead are still documented each year by landowners and agencies, and most tributaries to the Napa River appear to be well seeded with juveniles (EcoTrust, 2001). Despite reduced populations, the Napa River watershed is considered one of the most significant anadromous fish streams within San Francisco Bay (Leidy et al., 2005) (RCD, 2005)." (Corps 2006b)

York Creek is known to support Central California Coast steelhead. Habitat surveys by NOAA Fisheries Service and CDFG, completed during the summers of 2000 and 2002, found abundant steelhead downstream of the dam and resident rainbow trout above the dam (Corps 2006b). Snorkel surveys conducted in the summer of 2004, by Napa County RCD, found moderate densities of juvenile steelhead and larger resident rainbow trout in most reaches (Corps 2006b). In summer 2005, as part of the York Creek Diversion Modification project, a 120-foot reach of stream channel was dewatered to facilitate construction. Sixty-four steelhead were captured within the work area and relocated (PCI 2004). In summer 2006, the Upper York Creek Reservoir was dewatered to facilitate routine maintenance and sediment removal above the dam. In a 500-foot reach of stream channel, 12 rainbow trout were captured and released upstream (PCI 2006b). There are no reports of stocking trout in York Creek by CDFG (Corps 2006b).

Based on habitat data collected as part of the Central Napa River Watershed Plan by Napa County RCD, York Creek has been identified as one of the "most significant spawning and rearing streams for steelhead within the Napa Basin" (NCRCD 2005). The upper reaches of the watershed, above the dam, "offer excellent spawning and rearing habitat, and creating access to these areas will greatly benefit the overall steelhead population" (NCRCD 2005). As a result of dam removal, approximately 1.7 miles (8,855 feet) of aquatic habitat could become available to CCC steelhead. Newly accessible steelhead habitat would include 0.30 acres of pool, 0.33 acres of flatwater, and 0.85 acres of riffle habitat, totaling 1.5 acres. This reach could potentially support between 825 and 1,811 juvenile steelhead annually, under current habitat conditions (Corps 2006b).

Lower York Creek Reservoir. In addition, non-native upland vegetation, including ruderal vegetation around Upper York Creek Reservoir, ruderal and non-native grassland around Lower York Creek Reservoir, and ruderal vegetation at Spring Mountain Vineyard, will be impacted during maintenance activities.

Mitigation BIO-NP12:
Adverse but less than significant impacts do not require mitigation; however, to avoid and/or reduce the potential for adverse impact to vegetation, the City of St. Helena shall implement protective measures:

- Vegetation removal will be minimized to the maximum extent practicable during all work activities. Grading limits will be clearly flagged to minimize disturbance from construction equipment.
- Upland native trees greater than 12 inches diameter at breast height that are removed as a result of maintenance dredging activities will be replaced at a minimum 1:1 ratio with equivalent native species. All propagules used for native plantings will be obtained from local nursery stock, if available.
- All disturbed areas will be revegetated with native plantings and/or a native seed mix as soon as practicable to minimize erosion and recruitment of invasive non-native plant species. Best Management Practices that avoid dispersal of invasive non-native plants will be used, including using only certified, weed-free materials dominated by native species for erosion control and revegetation.
- Any tree removal associated with maintenance dredging will comply with the City of St. Helena's Tree Ordinance.

Significance after Mitigation. Less than significant.

3.3.4 Preferred Alternative - Small Notch: Impacts and Mitigation Measures

The Small Notch Alternative is designed to restore the ecological connectivity between habitats upstream and downstream of the dam and provide for a natural sediment transport system. Central California Coast (CCC) steelhead migration would be restored to 100% effectiveness[19]. This alternative allows for the removal of 95% of sediment behind the dam reducing the threat of future sediment releases and removal of 72% of the actual dam material. A 23-foot wide natural stream channel would be restored, and 2.0 acres of streambank (0.4 acres), terrace (0.5 acres), and riparian zone (1.1 acres) would be revegetated. Approximately 1.7 miles (8,855 feet) of aquatic habitat would become available to CCC steelhead including 0.30 acres of pool, 0.33 acres of flatwater, and 0.85 acres of riffle habitat, totaling 1.5 acres. This reach could potentially support between 825 and 1,810 juvenile steelhead annually under current habitat conditions (Corps 2006a).

The existing spillway adjacent to Spring Mountain Road would remain intact and be filled with sediment from the project site. This provides the most geologically stable alternative that allows for the restoration of York Creek's natural hydrologic functions. Notching the dam would generate approximately 40,000 cubic yards of materials (Corps 2006a). Materials not utilized during ecosystem restoration will be reused or disposed of at LYCR, SMV, and/or Clover Flat Landfill. Materials deposited at LYCR would impact 0.22 acres of freshwater wetland habitat, 1.2 acres of open water habitat, and less than 0.1 acres of foothill pine-oak woodland, non-native

[19] 100% effectiveness means unobstructed fish passage upstream and downstream of the project site.

Geomorphology

York Creek is predominantly classified as a moderately entrenched, riffle dominated, relatively stable, gravel/cobble substrate channel (Figure 12) (NCRCD 2005). Channel features were measured in a reach upstream of UYCD, which represents natural stream conditions, to determine average channel width, as well as pool and riffle lengths (Corps 2006a, Appendix A citing Entrix 2002; PCI 2007a). Average pool and riffle lengths are 105 and 128 feet, respectively (Corps 2006a, Appendix A). PCI measured channel widths range 20-25 feet. In the lower reaches where the channel is not so constrained by the steep valley walls, the planform begins to meander, and a pool/riffle structure occurs (Figure 13). Through the City, York Creek maintains a fairly consistent width and depth except for one location adjacent to the Beringer Winery warehouse where a large gravel/cobble bar has formed and filled the channel (Figure 14).

Sediment Transport

The area above the dam represents 55% of the total watershed and 63% of the steep, sediment-source zone. The reservoir captures the entire coarse bedload and a portion of the suspended load produced in its 4.4 square mile drainage area – approximately 1,000-5,000 cubic yards of material annually depending on the rainfall and runoff patterns. An average hydrologic year with no unusual storm events delivers roughly 1,250 cubic yards of sediment to the reservoir. The winter of 2005/2006, an above-average year with the largest storm in recent history, deposited approximately 3,000 cubic yards of material behind the dam.

Throughout the watershed, the channel bed is composed of sediment ranging from sand to small boulders. The mean grain size is consistently very coarse gravel (32-64mm) to small cobble (64-128 mm), with low percentages of fines in the deposits (NCRCD 2005, PCI 2007a). Sediment deposited in the upper reservoir ranges from silt to small cobbles. Forty-one percent of the material by weight is made up of fine sediment (<2 mm) transported as wash and suspended load, while 30% is bedload composed of coarse gravel (16 mm) and larger particles.

Regulatory Environment

The upper watershed, including UYCD, is located within unincorporated Napa County. The lower reaches of York Creek are within the limits of City of St. Helena. The upper dam property, structure, and right-of-way were purchased by the City of St. Helena in 1922 (Corps 2006a). The City of St. Helena has the only pre-1914 appropriative rights to York Creek.

The City of St. Helena's Municipal Code Title 15, Chapter 15.52 Flood Damage Prevention provides for the establishment of areas of special flood hazard through scientific studies and mapping, as recommended by the City floodplain administrator. The floodplain administrator has the responsibility to manage activities that affect flooding and maintain the carrying capacity of channels within, or affecting properties within, City limits.

Final Environmental Impact Report for the City of St. Helena York Creek Diversion Modification Project

(State Clearinghouse No. 2003012099)

LEAD AGENCY:

City of St. Helena
Department of Public Works
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January 14, 2004

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Executive Summary

Introduction

This executive summary for the final environmental impact report (EIR) for the City of St. Helena's (City's) York Creek water diversion structure (Diversion Structure) modification project briefly describes the objectives and alternatives to the proposed project. Known areas of controversy and issues to be resolved in this EIR are summarized. A summary table is provided that identifies impacts of the proposed project and mitigation measures that would reduce significant impacts to a less-than-significant level.

Project Objectives

The York Creek Modification Project has been designed to meet the following objectives.

Primary objectives:

- Improve fish passage to provide unimpeded upstream adult passage and downstream juvenile passage; and
- Allow the City to maintain their water diversion per their water rights without entraining or impinging fish.

Secondary objectives:

- Maintain point of diversion;
- Low maintenance;
- Minimize environmental impacts; and
- Cost effectiveness.

Alternatives Evaluated in the EIR

This final EIR includes an analysis of the proposed project, lots of action alternatives, and the No-Project Alternative. Summary descriptions of each of the alternatives are provided below. Refer to Chapter 5, Alternatives Analysis, for a detailed evaluation of alternatives.

Proposed Project

The proposed project will be located at the site of the existing Diversion Structure. The proposed project will cut a trapezoidal notch in the Diversion Structure and construct a series of five boulder weirs to provide fish passage through the stream reach. An infiltration gallery will be constructed to maintain the City's water right for diversion to their Lower Reservoir without impeding juvenile outmigration past the diversion.

No-Project Alternative

For this analysis, the No-Project Alternative is defined as no modification to the City's existing diversion structure and diversion operation. Under this alternative, the City would take no action

to improve passage conditions at the diversion structure site or modify their diversion operation or schedule.

Fish Passage Alternatives to Boulder Weirs

The following fish passage alternatives to the proposed boulder weirs were identified:

Ladder over existing diversion dam. This alternative would involve affixing a permanent fish ladder to the diversion dam designed to pass target species and age classes under low to moderate flow conditions. The benefit of this alternative is that it would not require demolition of the historic diversion structure, though significant modification might be necessary. The limitations of this alternative include plugging with debris, frequent maintenance, potential downstream scour, and impact to a potential historic landscape.

Roughened bypass channel around river right. To reduce impacts to the historic structure, this alternative would propose construction of a channel to pass around the right side of the diversion dam. The bypass channel would be designed with adequate slope and roughness to provide passage under low to moderate flow conditions. In addition to reducing the impact to the historic structure, the bypass alternative would allow passage of fish and aquatic organisms as well as sediment and debris loads. In protecting the historic structure, the alternative would cause the creek use the bypass channel, thereby leaving the Diversion Structure dry under most flow conditions and changing the historic landscape. Additional impacts include greater riparian area disturbance and potential sediment load increase. This alternative is not compatible with either of the diversion alternatives at the existing Diversion Structure: the infiltration gallery and upgraded screen.

Water Diversion Alternatives to Infiltration Gallery at Dam

The following water diversion alternatives to the proposed infiltration gallery immediately upstream of the diversion dam were identified:

Upgrade screen at existing diversion. Because the current system has an inadequate screen, this alternative would retrofit the current diversion structure with a screen that meets NOAA Fisheries and DFG criteria for anadromous salmonids. The benefits of this alternative are that it would maintain the point of diversion and not require removal of as much of the historic diversion structure as the preferred alternative, though significant modification of the diversion box would be necessary. The limitations of this alternative include large riparian impact, frequent maintenance, and the need to bring in a power source for an automatic cleaning system.

Infiltration gallery upstream from the dam. Similar to the preferred alternative, this alternative would construct an infiltration gallery, but located it approximately 100 ft upstream of the existing diversion dam. Siting the infiltration gallery upstream would reduce the impact to the historic structure, though modification of the diversion box would still be necessary to connect the infiltration gallery to the pipeline to the Lower Reservoir. The drawbacks of this alternative include a larger construction impact area, construction of an access road, and the change to the point of diversion, which would affect the City's water right.

Upstream screen. Construction of a modern screen 80 ft upstream of the existing structure has the benefit of significantly reducing the impact to the historic structure. Water passing through the screen would continue down the creek and be diverted by the existing diversion box.

Modification of the historic structure would be required to repair of the leak through the dam's metal culvert and gate valve. In addition to the same limitations as the previous screen alternative, this alternative could also be considered a change in the point of diversion.

Known Areas of Controversy

Section 15123 of the State CEQA Guidelines requires that the summary of an EIR must identify areas of controversy known to the lead agency, including issues raised by public agencies and the public. No known areas of controversy have been identified regarding the proposed project as part of the Notice of Preparation process.

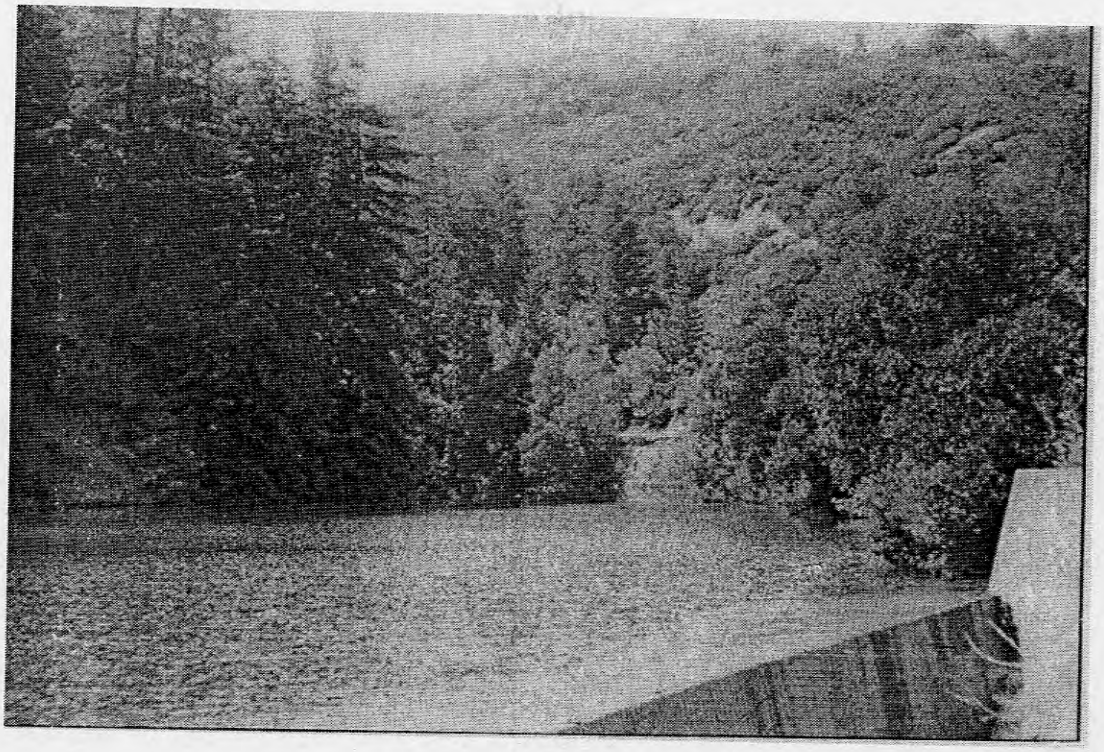
Issues to be Resolved

Section 15123 of the State CEQA Guidelines requires that the summary of an EIR must identify issues to be resolved, including the choice among alternatives and whether or how to mitigate significant effects of the proposed project. For the proposed project, the primary issue to be resolved is the unavoidable impact to the historic structure and landscape.

Summary of Impacts of the Proposed Project

Table ES-1 presents the impacts of the proposed project as analyzed in the final EIR, identifies the level of significance of each impact before mitigation, recommends mitigation measure, and identifies the level of significance after mitigation for each impact.

HISTORICAL RESOURCES EVALUATION REPORT FOR THE PROPOSED REMOVAL OF AN EARTHEN DAM AND DIVERSION STRUCTURE ON YORK CREEK NEAR THE CITY OF ST. HELENA IN NAPA COUNTY, CALIFORNIA



Prepared for the City of St. Helena

Anthropological Studies Center
Sonoma State University
Rohnert Park, California 94928

HISTORICAL RESOURCES EVALUATION REPORT FOR THE PROPOSED REMOVAL OF AN EARTHEN DAM AND DIVERSION STRUCTURE ON YORK CREEK NEAR THE CITY OF ST. HELENA IN NAPA COUNTY, CALIFORNIA

prepared by
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submitted by
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2-16-03

(signature) *(date)*

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February 2003

SUMMARY OF FINDINGS

The City of St. Helena, with assistance from the California Department of Water Resources (DWR), is proposing the removal of an earthen dam and a smaller associated diversion structure on York Creek alongside Spring Mountain Road near the town of St. Helena in western Napa County. The goal of the proposed undertaking is to restore aquatic habitat for Central California Coastal steelhead trout, a threatened species, by implementing structural improvements that will facilitate fish passage in two areas of the York Creek drainage. Because York Creek is considered jurisdictional "waters of the U.S.," the action requires the City to apply to the United States Army Corps of Engineers (USACE) Regulatory Branch for a 404 permit. This project is a federal undertaking, and its effects on cultural resources must be considered. To this end, a Historical Resources Evaluation Report (HRER) has been prepared to document the recordation and evaluation of structures on York Creek. It has been prepared to partially satisfy the requirements of Section 106 of the National Historic Preservation Act, as amended, and its implementing regulations (36 CFR 800, December 2000).

The Anthropological Studies Center (ASC) at Sonoma State University, Rohnert Park, California, conducted this study. The goal of this study was to inventory structures within the project's Area of Potential Effects (APE) and to evaluate their potential eligibility to the National Register of Historic Places (NRHP). The properties have also been assessed in accordance with Section 15064.5 of the California Environmental Quality Act (CEQA) Guidelines to determine if they are historical resources for the purposes of CEQA. Bright Eastman, Staff Architectural Historian for the ASC, conducted the field survey and historical research for properties within the project APE. The APE for this study is discontiguous, consisting of two related but separate features on York Creek alongside Spring Mountain Road—the Upper Reservoir and its associated structures, and the York Creek Diversion. These two properties have been formally evaluated in this report and appear to be eligible for the NRHP. They may also be considered historical resources for the purposes of CEQA and are eligible for listing on the California Register of Historical Resources pursuant to California Public Resources Code Section 5024.1(d) (1). Their significance is summarized below.

Upper Reservoir and York Creek Diversion

The St. Helena Upper Reservoir and York Creek Diversion appear eligible for the National Register under Criterion A, at a local level of significance in the area of community planning and development. With a period of significance of 1900, the reservoir and downstream diversion dam are important features of infrastructure for the town of St. Helena in the early 20th century. This water collection and conveyance facility was essential for the growth and development of the town in the early 20th century because it extended the city's water storage and distribution system, making more water service individually available to the host of new commercial and residential properties that were being built in addition to assuring more water available for fire protection.

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INTRODUCTION

Project Description

The City of St. Helena in Napa County (Figure 1) is undertaking planning level studies for the purpose of improving habitat for Central California Coastal steelhead trout, a threatened species, in York Creek, a tributary to the Napa River. The City's efforts will focus on implementing structural improvements to facilitate fish passage and restore aquatic habitat in two main areas of the York Creek drainage alongside Spring Mountain Road. The City is acting as the lead agency and partnering with DWR on modifications to the York Creek Diversion Structure and in a separate effort the City is partnering with the U.S Army Corps of Engineers (USACE) on modifications to the Upper York Creek Dam.

The project proposed for the York Creek Diversion Structure involves modification and rehabilitation of the structure to improve fish passage in York Creek while maintaining the City's ability to divert water to their Lower Reservoir. The proposed project would involve introduction of fill to York Creek and, because York Creek is considered jurisdictional "waters of the U.S.," the action will require that the City apply to the USACE Regulatory Branch for a 404 permit (Clean Water Act, 33 U.S.C.1344).

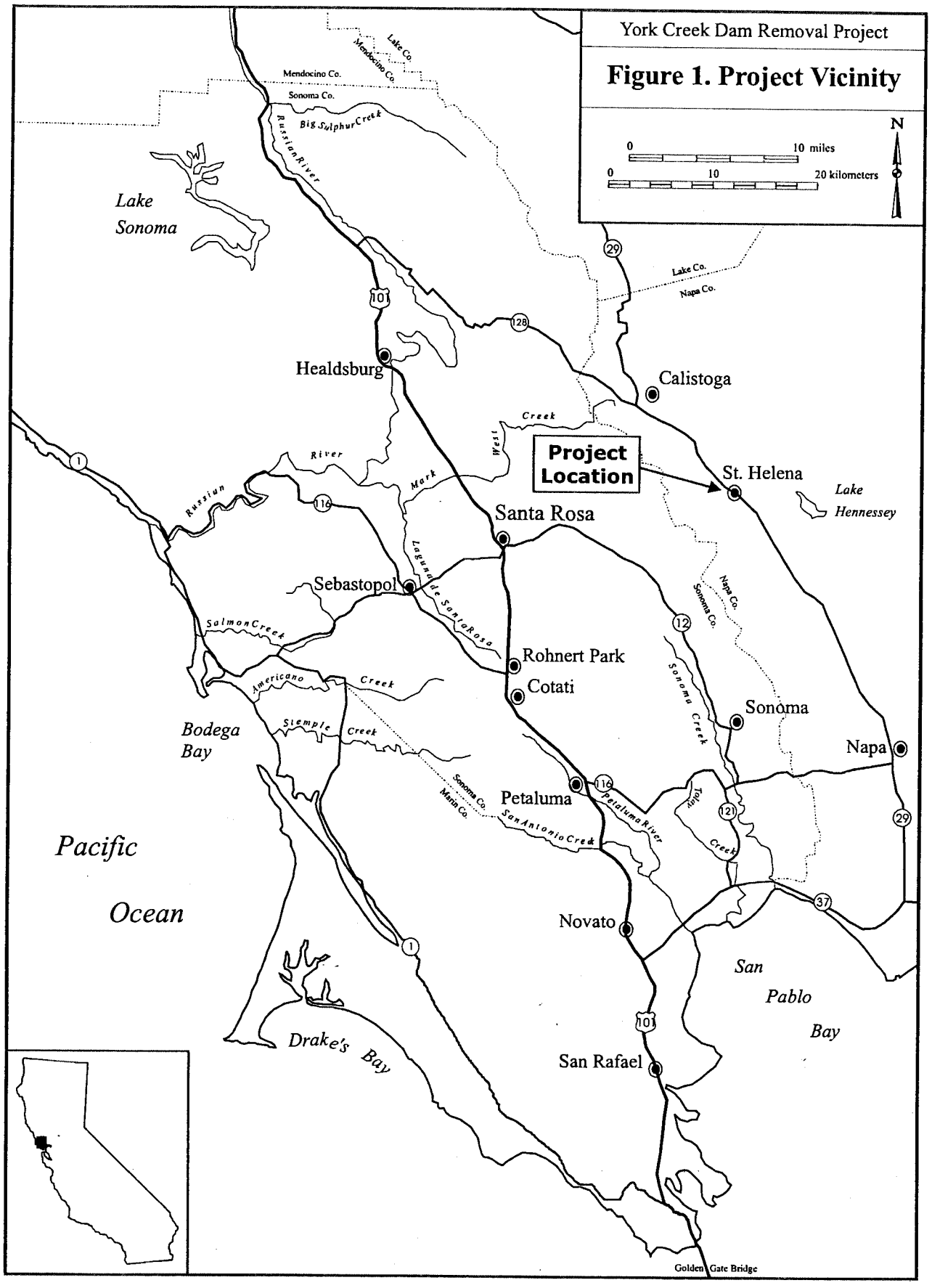
Work on the Upper York Creek Dam will proceed under the USACE Continuing Authorities Program Section 206 of the Water Resources Development Act of 1966.

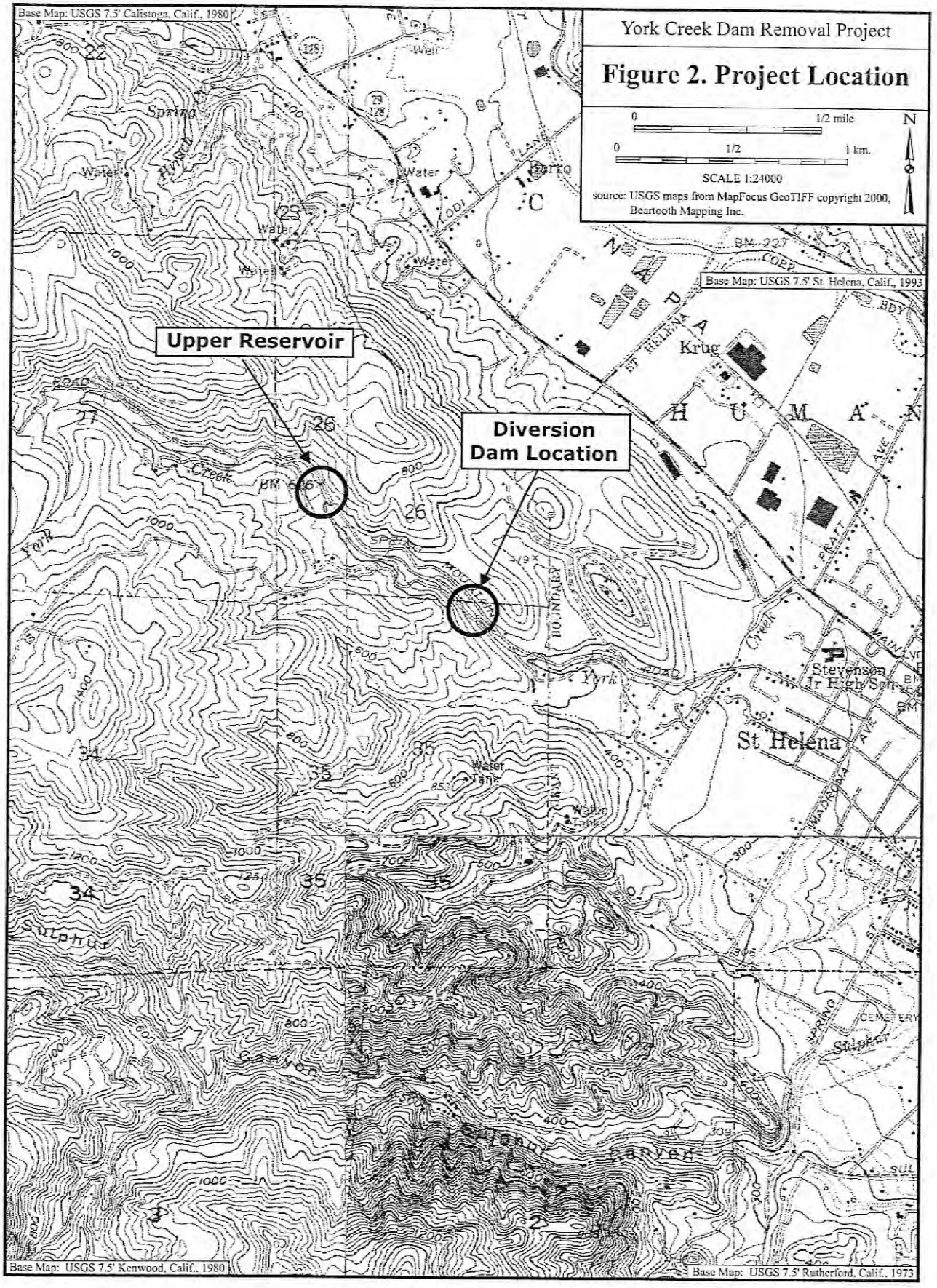
+ Section 206 provides for the restoration of aquatic habitat and will allow the City to achieve its goals of riparian restoration and facilitation of fish passage where appropriate. Under Section 206 the Corps will study the area, identify and implement a restoration project.

In both areas the planning process to define potential restoration measures may recommend modifications or removal of City owned structures. The potential removal or modification of these structures warrants a Historical Resources Evaluation Report (HRER), which will document the recordation and historical evaluation of the structures. The HRER has been prepared to partially satisfy the requirements of Section 106 of the National Historic Preservation Act, as amended, and its implementing regulations (36 CFR 800 December 2000).

The purpose of the studies documented in this HRER is to inventory structures within the area of potential effects (APE), and to evaluate their potential eligibility to the National Register of Historic Places (NRHP). In accordance with Section 15064.5 of the California Environmental Quality Act (CEQA) Guidelines, the structures were also assessed to determine if they are historical resources for the purposes of CEQA. Two properties have been evaluated in this document. Details regarding their evaluation may be found on the Department of Parks and Recreation (DPR) form 523 included as an "Appendix".

The project APE encompasses all properties at least 50 years of age that are subject to potential effects of the proposed undertaking, resulting in a change to their historic character. National Register criteria, as well as California Register criteria found in PRC 5024.1, use a general minimum age of 50 years as the cutoff date for potentially eligible resources. The APE for this study is discontiguous, consisting of two related structures on York Creek alongside Spring Mountain Road—the Upper Reservoir and the York Creek Diversion (Figure 3). The streambed between the two resources is excluded from the APE because


York Creek Dam Removal Project
Figure 1. Project Vicinity
N
0
10 miles
0
10
20 kilometers
Lake Sonoma
Mendocino Co.
Sonoma Co.
Lake Co.
Mendocino Co.
Big Sulphur Creek
Russian River
Lake Co.
Napa Co.
Healdsburg
Calistoga
Project Location
St. Helena
Lake Hennessey
Santa Rosa
Sebastopol
Laguna de Santa Rosa
Mark West Creek
Russian River
Salmon Creek
Americano Creek
Stemple Creek
Rohnert Park
Cotati
Sonoma
Napa
Sonoma Creek
Napa Co.
Sonoma Co.
Bodega Bay
Petaluma
Sonoma Co.
Marin Co.
San Antonio Creek
Petaluma River
Tolay Creek
Pacific Ocean
Novato
San Pablo Bay
Drake's Bay
San Rafael
Golden Gate Bridge


York Creek Dam Removal Project
Figure 2. Project Location
0
1/2 mile
0
1/2
1 km.
N
SCALE 1:24000
source: USGS maps from MapFocus GeoTIFF copyright 2000, Beartooth Mapping Inc.
Base Map: USGS 7.5' Calistoga, Calif., 1980
Base Map: USGS 7.5' St. Helena, Calif., 1993
Base Map: USGS 7.5' Kenwood, Calif., 1980
Base Map: USGS 7.5' Rutherford, Calif., 1973
Upper Reservoir
Diversion Dam Location
Krug
H U M A N
St Helena
Stevenson Jr High Sch
Spring
Water
Weir
Harto
York
Creek
Sulphur
Canyon
Sulphur
Canyon
Sulphur
CEMETERY
BOUNDARY
ST HELENA
MAIN
PRATT
MADRONA
COOPER
BM 227
BM 606
Water Tank
Water Tanks

historical research indicates the streambed was never modified for construction or maintenance of the original dam and diversion system. This observation was corroborated by a November 2002 interdisciplinary physical assessment survey of the streambed between the two structures on York Creek (Entrix, Inc. 2002). The Upper Reservoir encompasses an area roughly 700 feet long by 180 feet wide. It includes a concrete spillway and collecting pond lying adjacent to the south side of Spring Mountain Road, the interior of the Upper Reservoir, the earthen dam, the outfall, and the original concrete spillway located on the south end of the dam. The APE for the Diversion dam encompasses the streambed and embankment, a stone masonry dam with concrete capping, corrugated metal pipe and slide gate, intake box, and concrete apron. Structures that were subject to study are shown on the project location map (Figure 2) and on the APE map (Figure 3).

Research Methods

Bright Eastman, Staff Architectural Historian for the Anthropological Studies Center (ASC), undertook the study, which included field survey and recording the resources, conducting historical research, and preparing the report. She received assistance from City of St. Helena employees and staff from the Department of Water Resources. Historical research was conducted at the Napa County Assessor/Recorder's Office, the Napa County Library, the St. Helena Public Library, Napa County Historical Society, and the City of St. Helena Planning Department. Mr. Robert Brownell, Water Foreman for the City of St. Helena, provided Ms. Eastman with detailed information on the history and location of the City's water system, with special emphasis on the Upper Reservoir and Diversion structure. The California Department of Water Resources supplied photographs, mapping, and historical documents pertaining to the structures on York Creek.

Prefield research also consisted of a thorough review of the following resources:

- *National Register of Historic Places* (National Park Service [NPS] 1990 and current updates; State Office of Historic Preservation, Sacramento, California, n.d.);
- *California Register of Historical Resources* (State Office of Historic Preservation, Sacramento, California, 2001);
- *California Inventory of Historic Resources* (California Department of Parks and Recreation, Sacramento, California 1976);
- *California Historical Landmarks* (State Office of Historic Preservation, Sacramento, California, 1996);
- *California Points of Historical Interest* (State Office of Historic Preservation, Sacramento, California, 1992, and updates);
- *Napa County Historical Resource Inventory, St. Helena Volume* (Napa County Landmarks, Inc., Napa, California, 1978);
- *Survey of Surveys* (State Office of Historic Preservation, Sacramento, California, 1989);
- *Historic Property Data File for Napa County* (2001)

- Miscellaneous local histories (e.g., Menefee 1873; Slocum, Bowen & Co. 1881; Gregory 1912; Loeber 1955; Wichels 1976).

In the course of historical research, themes for the project area emerged that revolve around initial Anglo-European settlement of the Napa Valley, agriculture, transportation, water issues, and community development. These themes produced the contexts in which the properties were evaluated for their historic significance.

HISTORICAL OVERVIEW

General History

Napa County

What is now the County of Napa was, after 1822, governed by Mexico, whose Northern Military Department headquarters was established in the pueblo of Sonoma to the west in the 1830s (Menefee 1873:32). The Mexican government made concessions, or grants, of tracts of land for grazing throughout California, and 15 such land grants—called "ranchos"— were claimed in the Napa territory (Beck and Haase 1988:24). The first American to be given a Mexican land grant from Sonoma's commandant, General Mariano Vallejo was George Yount in 1836. Vallejo's son-in-law, Dr. Edward T. Bale, was given the Carne Humana Rancho in the north end of the valley. Yount and Bale were among the early American and European fur trappers who decided to settle in northern California during the period of Mexican occupation (Loeber 1955:2). Growing American interest in Mexico's northern territories brought settlers in ever-increasing numbers, ultimately ending in war between the United States and Mexico in 1846. The war concluded in 1848 with the signing of the Treaty of Guadalupe Hidalgo, in which Mexico ceded California and other territories to the United States (Bean and Rawls 1988:77-79).

The county of Napa was organized into 828 square miles in 1851. A considerable portion of the north half of the county was cut in 1861 and reconfigured as part of Lake County. The total area of Napa County today is 450,000 acres, consisting of three parallel valleys running in a northwesterly and southeasterly direction, parallel with the Pacific Coast (Menefee 1873:34). The Napa River runs through the widest valley, emptying in the south into San Pablo Bay.

Transportation, 1850 – 1900

The mild climate of the Napa Valley—warmer than the coast but not as severe as the Central Valley—made it very attractive to immigrants and migrants seeking farm and ranch land in the general vicinity of San Francisco. There were also mining, quarrying, and lumbering opportunities in the county, and ventures associated with its many hot springs. Napa River was the ideal port from which to ship local products to San Francisco and to receive regular supplies of goods and services. A town grew up on the river at the south end of the Napa Valley that became Napa City in 1848. The first steamer ran between Napa and San Francisco in 1850 (Menefee 1873:52). The most passable major coach road in the county connected Napa City with the hot springs resort town of Calistoga, a distance of about 32 miles. By the late 1870s, the county road was receiving regular maintenance and graveling to at least as far as St. Helena. Regional railroads and branch lines were crucial to the development of the state after 1860 (Bean and Rawls 1988:162). The Napa Valley

Railroad Company was chartered in 1864, and by 1867 its line extended all the way to Calistoga. Unable to sustain adequate financing for its railroad, the county's interest passed into the hands of the California Pacific Railroad, and subsequently into those of the Central Pacific (Menefee 1873:56).

Community Development in the Napa Valley, 1850 - 1900

With Napa City as the southernmost gateway to the Napa Valley, the villages Yountville and St. Helena grew up along the major county road that connected Napa City with Sam Brannan's hot springs resort town of Calistoga. From the 1860s through the turn of the century, agriculture was the major impetus for community development in Napa County as elsewhere in California. Initially, cattle were raised in the valley for the hide-and-tallow trade within the Mexican rancho tracts. As agriculture took over the most arable lands in the Napa River valley, livestock raising continued in the smaller valleys and hills. Improvements in agricultural production along with a general upswing in the state and regional economy in the 1880s motivated landowners to subdivide and sell off portions of their rancho tracts to ambitious agriculturalists who were eager to establish themselves in a locale that was winning a reputation for high profits (Bean and Rawls 1988:188). The small communities of Oakville and Rutherford were established around the railroad depots, where local farmers loaded their products for shipment.

Agriculture in mid-19th century Napa County mirrored the statewide trends in production (Bean and Rawls 1988:187). Wheat grew favorably in the center of the Napa Valley, along with barley. These, and other crops were sold and shipped to grain warehouses that lined San Pablo Bay. St. Helena rapidly grew in importance as the "vineyard district" of Napa County, where large amounts of wine and brandy were produced by resident growers of American, English, German, and Swiss descent (Menefee 1873:58). In the 1880s, the vine pest *phylloxera* practically decimated the county's vineyards, particularly in the southern portion of the valley. With importations of European root stock resistant to the disease, the vineyards bounced back into healthy production by the century's end (Loeber 1955:3).

Economic successes led to growing social stability and the development of California's cities and towns (Bean and Rawls 1988:162). Schools, churches, commercial buildings, hotels, and many more residences were erected in Napa City and the smaller towns in the county between the late 1860s and 1870s. Banks, city halls, and newspapers were established, as were fire departments and businesses that maintained the necessary infrastructure to support the new towns. Among these were gaslight companies, cemetery associations, and private water companies that harnessed the waters from local creeks, springs, and wells to sell to the public (Menefee 1873:109-111). By the early 1900s, water works and municipal water utilities had been established in all Napa County towns (Tortorolo, 1978:2).

St. Helena, California, 1848 – 1900

The city of St. Helena in Napa County, California, was established in a portion of the Carne Humana Rancho, a Mexican land grant ceded in 1839 to General Mariano Vallejo's English son-in-law, Dr. Edward T. Bale. The earliest buildings erected in the locale were Dr. Bale's adobe dwelling and a large gristmill he built in 1846. In 1848 a two-story wood-framed house was built for Dr. Bale and his family next to the mill (Loeber 1955:4). A village began to grow up in the vicinity of Bale's gristmill in 1850, and in 1876, the town of

St. Helena was incorporated, taking its name from Mount St. Helena at the north end of the Napa Valley. The new town was about two miles square, with the Napa River forming the eastern boundary. Agriculture, specifically winemaking, was the major economic activity in and around St. Helena, as it had been since the early 1850s. Pioneer vintners like Charles Krug, G.B. Crane, and the Beringer Brothers operated large wineries that produced thousands of gallons of wine, earning St. Helena the title of "Table Wine Growing Center of the World" (Loeber 1955:7).

The town developed all of the institutions and amenities that are characteristic of a successful settlement where families could be raised. The first school was erected in 1856 and the first private high school was established in 1870. St. Helena's first church was the Baptist Church built in 1858, and by 1883, the town boasted a Methodist Church, a Presbyterian Church, a Seventh Day Adventist Church, a Catholic Church, and an Episcopal Church. The St. Helena Cemetery Association was formed in 1870, and the first bank was created in 1877. St. Helena's newspaper, the *St. Helena Star*, was started in 1874. A volunteer fire department was organized in 1878 and remains volunteer to this day. The first electricity in St. Helena was generated in 1884 and the first gas company was created in 1885. St. Helena's telephone and telegraph system was in operation by 1892 (Loeber 1955:8-9). By about 1900, the utilities and city streets were installed and well maintained, enhancing the quality of life for the growing number of residents who were building homes in St. Helena.

Water for agriculture and town use was made available from wells that tapped into a large aquifer that flowed underneath the center of the town (Loeber 1995:9). In 1871, a small dam and reservoir were built on Hudson's Creek (later York Creek). From a small- diameter iron pipeline, a modest flow of water was delivered to nearby vineyards, ending its journey in a central receptacle in town (R. Brownell 2002, pers. comm.). In the 1870s, reservoir and dam projects were undertaken that would make St. Helena the first town in Napa County to have city water service.

The First Dam and Reservoir – 1871

In addition to a large and deep aquifer that flowed from the northwest in a southeasterly direction underneath the center of St. Helena, land owners along the creeks and springs in the vicinity retained riparian rights to support their agricultural operations (R. Brownell 2002, pers comm.). Recognizing the need to conserve and share water to sustain the economic interests of the community, winegrowers John York, Mr. and Mrs. Charles Krug, David Hudson, and Robert Hastie assigned their water rights and title to local winemaker, David Fulton, in 1869 for the purposes of constructing a small reservoir to contain "Hudson's Creek," later named for John York. John York eventually leased Fulton his rights because he had already built a flume some distance above the dam site that irrigated his property. David Fulton had a winery in town and was noted for his patented one-horse plow, which improved vineyard cultivation without damaging the delicate vines (Wichels 1976:34).

The reservoir was to hold the creek's waters for diversion into the vineyards of the agreeing parties above the town. For the amount of $44.00, Fulton hired local builder R. Teeple to construct the rock dam and reservoir that was supposed to be 10 feet deep, 20 feet wide, and about 90 feet in length. The amount of water each party was to receive was "such quantity as will flow in a 1 inch pipe, and each one to share in the expense involved."

(Wichels 1976:35). The small reservoir was completed in April 1871. David Fulton passed away four months later and his water rights were allocated to S.A. Hanna (Wichels 1976:50)

St. Helena Water Company - 1877

The population of St. Helena rose in the 1870s with the promise of profits from winemaking, in addition to development of services and infrastructure. By 1874 the town Trustees were regularly warning residents to conserve water due to the threat of fire that could ravage the town's many buildings. Residents and town officials called for a ready supply of water to the city that could be used to effectively fight fires, as individual wells limited firefighting to bucket brigades (*St. Helena Star* 1874). Their pleas were finally answered in July of 1877, when a group of local wine growers with lands along York Creek filed articles of incorporation as the St. Helena Water Company, organized for the purpose to supplying the town of St. Helena with fresh water (*St. Helena Star* 1877). Land was deeded to the company for the project by its members, some of whom had participated in the first reservoir project in 1871 (Napa County Deeds 1878 a,b). Its early directors all had lands adjacent to York Creek. Seneca Ewer was president and Charles Krug became secretary to the company. Jacob S. Beringer and G.K. Gluyas were also among the original members. The only change in the board of directors came in 1879, when D.B. Carver succeeded John York (Slocum, Bowen & Co. 1881:344). Capital investment in the project amounted to $25,000, which was sold in shares of $10.00 each. St. Helena residents were encouraged to invest in the city's new water company to insure its success (*St. Helena Star* 1877).

The Lower Reservoir - 1878

Initial plans for the waterworks were to take up water from York Creek in a flume that that led 1/4 mile to a reservoir. The reservoir site chosen by the company was located on the former lands of Charles Krug; the reservoir was to spread over an area of 10 acres and was to be contained by an earthen dam built across a ravine. Originally, plans called for the dam to be 40 feet high and approximately 320 feet long, giving the reservoir a capacity of about 20,000,000 gallons (Slocum, Bowen & Co. 1881:344). By May of 1878, construction on the reservoir was underway. Water engineer T.B. Lewis of San Francisco was contracted by the company to complete the work. It was estimated that the dam would end up being 24 feet high and over 260 feet long, and about 25 feet wide on top. Thirty thousand "loads" of earth were removed from the reservoir site to build the base of the dam, which was 120 feet long and calculated to withstand a height of 50 feet. A certain amount of water was then allowed to flow into the reservoir to "settle the work." The reservoir was connected to the water source by a 12-inch, cast-iron delivery pipe that was 140 feet long and "goose-necked" four feet high inside the reservoir to raise the water flow above the sediment. A stop valve was placed on the outlet pipe that was connected to the main that led into town (*St. Helena Star* 1878a).

With construction of the reservoir moving at a satisfactory pace in August of 1878, the town Trustees planned to install six 3-inch, single-discharge hydrants on Main Street strategically placed throughout the town—one at the "hay scales," one at the corner of Spring and Main Streets, one opposite Hunt Avenue, one at the *St. Helena Star* office, and three on private property. The distance between the first four hydrants would be only 300 feet so that the densely developed portion of the town would be well supplied with water for any necessity. Town Trustees also approved the purchase of 150 feet of 3-inch hose for each hydrant, so that in the event of a fire in the commercial district of town, four streams of water

could be brought to bear upon the flames. Eager to complete the hydrant installations by the time the reservoir was complete, the town Trustees urged citizens to organize a volunteer fire department so that their town would be "one of the safest from fire on the coast" (*St. Helena Star* 1878d).

Within two weeks after the installation of fire hydrants in town, the dam and reservoir were finished on schedule at the end of August 1878, prior to the onset of the rainy season. The *St. Helena Star* heralded it as an achievement that "marked an epoch in the history of St. Helena, and will long be remembered as the dawn of a new prosperity, the herald of accumulating years filled with all that goes towards beautifying our town and rendering its homes among the most pleasant in our favored valley" (*St. Helena Star* 1878b). Completed at a cost of $3,909.58, the dam was 223 feet in length, 55 feet high and 21 feet wide on top. Trenches 2 feet deep were dug from the reservoir to town, in which 7,080 feet of pipe was laid. From the reservoir, 12-inch-diameter pipe was used, narrowing to 6-inch pipe in town. Nearly 2,000 feet of lateral pipe was also laid, resulting in a total length of 9,000 feet of water-conveyance pipes. Waterfall from the reservoir to town was 170 feet, providing enough water pressure at the hydrants in town to throw water a distance of 105 feet. St. Helena let it be known throughout the county that it was the first town in Napa County to provide city water (Slocum, Bowen & Co. 1881:335).

The total cost of the project came to around $26,000, and increasing numbers of subscriptions from townspeople helped to offset the cost (*St. Helena Star* 1878b). By September of 1878, the St. Helena Water Company had set temporary water rates for its customers: $1.00 a month for dwellings, and $1.50 for tubful at the public baths. Hotels were charged $2.50 for baths and sprinkling, and commercial window and sidewalk washing came to $0.02. In stables, water for horses and cows was $0.20 for each animal, with a rate of $0.15 each for over five head. Irrigation with the company water was charged at $0.75 for 50 to 100 square yards, and $1.00 for 150 square yards and over (*St. Helena Star* 1878c).

As the population of St. Helena increased, so did its needs for more water. The St. Helena Water Company met this demand with improvements to the reservoir and dam in 1882. By June of that year the base of the dam was strengthened to withstand raising its height 20 feet (*St. Helena Star* 1882a and b). Improvements continued the following year, when the company contracted engineer and builder James R. Davis to raise the dam another 10 feet and lengthen it by another 150 yards (*St. Helena Star* 1883).

The Upper Reservoir and Diversion– 1900

Throughout the latter decades of the 19th century, the St. Helena Water Company reservoir served the needs of the town, providing water for commercial and residential purposes. By 1899, however, the original water delivery system was stretched to capacity with the increasing number of homes and businesses that were being built in St. Helena. Seeing the need to extend the town's water delivery system to match its expanding development, water company president, F. Seneca Ewer, announced his plans to build a new water works on York Creek above the original reservoir, and about a mile-and-a half from the center of town (*Napa Daily Journal* 1899). On land leased from the St. Helena Water Company, Mr. Ewer was granted a franchise to build a second water works that would deliver water directly to St. Helena consumers. He engaged an engineer named J. M. Graham to complete the work. The *St. Helena Star* reported that the site selected for the project was "1000 to 1200 feet from the head of York Canyon on York Creek," where a dam

about 45 feet high, 250 feet thick at the base, tapering to a thickness of 14 feet was to be constructed. Plans called for a sizable reservoir to be excavated behind the upstream face of the dam, and the water passing through a sluice gate would run down York Creek for a distance of around one half to three quarters of a mile over rocks in the streambed that were supposed to act as a natural filter. Thus relieved of much sediment, the clear water could then be diverted from the creek and conducted into a storage and distribution system that led directly consumers in town (*St. Helena Star* 1900).

When it was completed in 1900, the upper reservoir on York Creek covered three acres and had a 10,000,000-gallon storage capacity. The earthen filled dam was 35 feet high and 140 feet long on the crest. Both faces were covered with riprap stone and there was a culvert and 6-inch draw-off pipe through the center of the dam with a sluice gate and screw gear for regulating the discharge from the reservoir. A concrete spillway was built alongside the dam with a wooden flume that carried overflow into the creek. The diversion structure was built a little over half mile below the upper reservoir, consisting of a smaller dam of rubble stone masonry that was capped with concrete. A concrete, screened intake box diverted water restrained by the little dam into a 16-inch riveted steel pipeline that ran underground for 1609 feet to the lower reservoir where it emptied into a large 30,000 gallon redwood tank. Water from the York Creek diversion was stored in the redwood tank that connected directly to the main pipes leading into town. Excess water flowed into the lower reservoir (Hoar 1922:6-7, 19) (Figure 4).

St. Helena had grown in into a busy agricultural town in the early 20th century and, in 1922, the town purchased all 39 ½ acres of land owned by the St. Helena Water Company that included the upper and lower reservoirs, rights-of-way, and conduits (Napa County Official Records 1922). Now having its own municipal water utility, St. Helena's water collection and storage facilities were more than adequate, and the distribution system consisted of the steel riveted main pipes laid in 1878 that ran 3600 feet to the central distribution network that began just north of the business section of Main Street. At the time the town acquired the system in 1922, the network of water mains ranged from 8 to 2-inch pipeline that totaled 45,000 feet. There were a total of 46 four-inch hydrants and four 2-inch hydrants that furnished fire protection. All businesses and residences had their own water supply and anticipated growth was to be accommodated with short extensions in the established system (Hoar 1922:8) (Figure 5).

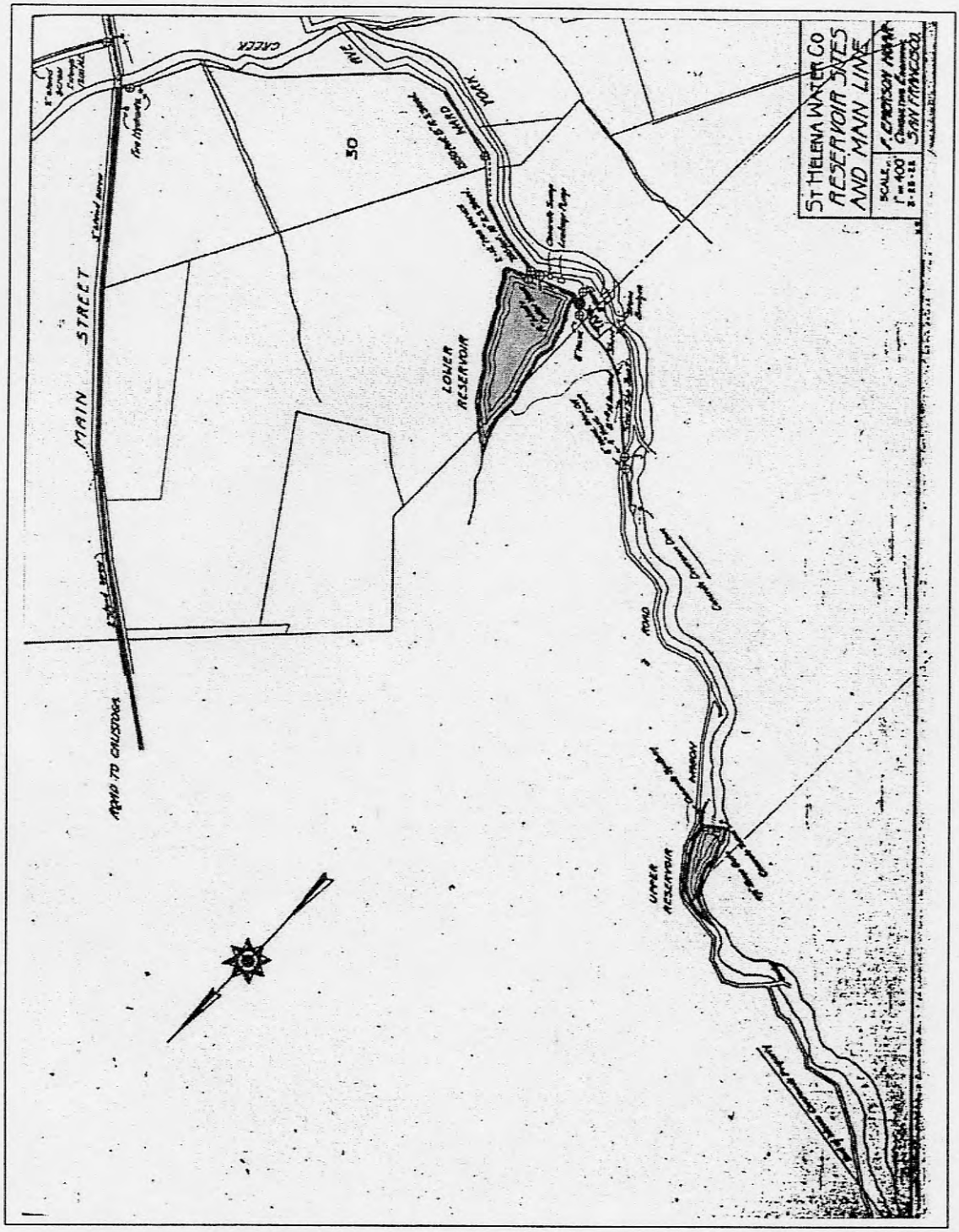


Figure 4: Schematic drawing of the Upper Reservoir, Diversion dam, and the Lower Reservoir in 1922 (*F. Emerson Hoar 1922).*

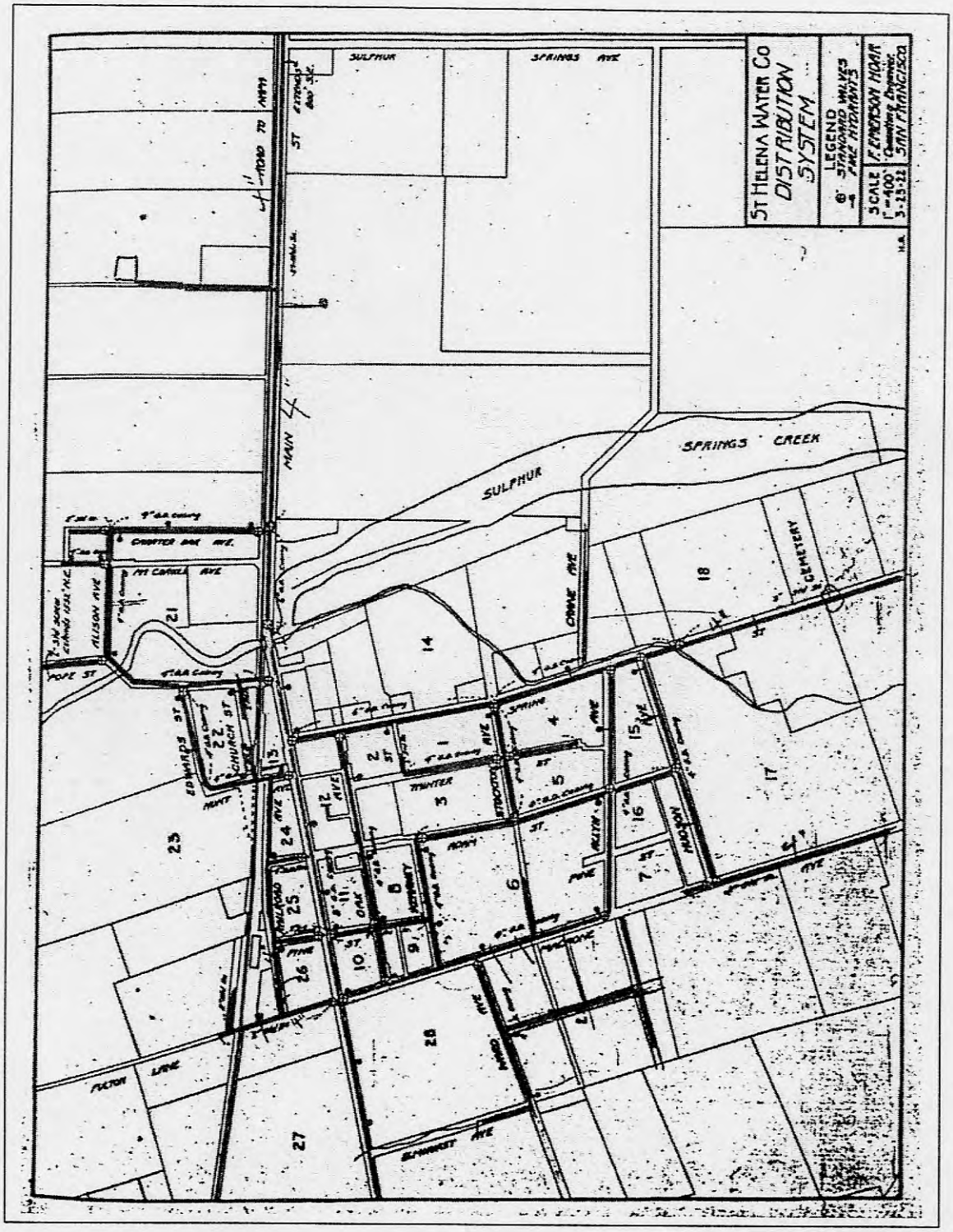


Figure 5: Map showing the St. Helena water distribution system in 1922 (*F. Emerson Hoar 1922).*

FIELD METHODS

Bright Eastman conducted field surveys on 12 and 27 September 2002. Robert Brownell, the City of St. Helena's Water Foreman, accompanied Ms. Eastman on 27 September because part of the survey included viewing the Upper Reservoir and Diversion on York Creek in relationship to other features of the city's water-conveyance system, both historic and modern. Features viewed in the broader survey included the Lower Reservoir built in 1878, the former fluoridation equipment from the 1940s, the chlorination station constructed in the 1950s, intake valves, and locations on Main Street where valves from the original fire hydrants are still present. Mr. Brownell also corroborated the identification of some of the original riveted conveyance pipes laid by the St. Helena Water Company in 1878.

The Upper Reservoir and Diversion dam were recorded on the California Department of Parks and Recreation Primary Record and Building, Structure, and Object Record forms (DPR 523A and 523B; Appendix A). Photographs taken by Department of Water Resources environmental staff were used in the recording process. The Department also supplied the topographic mapping of the structures that have been incorporated into this report.

DESCRIPTION OF THE RESOURCES

The Upper Reservoir

The City of St. Helena's Upper Reservoir is located on York Creek approximately 1.5 miles upstream from the city along the south side of Spring Mountain Road. The reservoir is abandoned and covers an area approximately 420 feet long and 120 to 180 feet wide. A 6-foot stand-pipe outlet is situated in the bottom of the reservoir, approximately 38 feet behind the dam (Figure 7). Earth excavated in 1900 for the reservoir was used to construct the earthen-filled dam, which was faced on both sides with basaltic fieldstone riprap. The dam rises to a height of 50 feet, with a crest elevation of 620 feet above mean sea level. It is 140 feet long and is about 30 wide on top. The intake and outlet pipes are located at the center of the base of the dam. The remains of the original iron screw gear for the slide gate lies on top of the dam near the southernmost end. A keystone arched stone culvert surrounded by roughly cut stone masonry abutments is located at the center of the outfall on the downstream side of the dam (Figure 6). It measures 3 feet in diameter and contains a 9-inch, riveted pipe and discharge valve. The original concrete spillway extends from the top of the south end of the dam to the creek bed below. A second spillway 270 feet in length was constructed in 1933 adjacent to the north end of the dam alongside Spring Mountain Road. It is concrete-lined and flows downstream to a scoured pond at the north side of the creek below the dam outlet. The reservoir and dam have not been reconstructed or altered in a major way since initial construction in 1900. Routine repair and maintenance of existing parts were performed on the structure in the 1970s. Modifications to bring the dam into compliance with safety standards were completed in 1985, but these alterations did not change the historic appearance of the structure, or negatively affect its original setting, feeling, or association with the original design (DWR 2003).

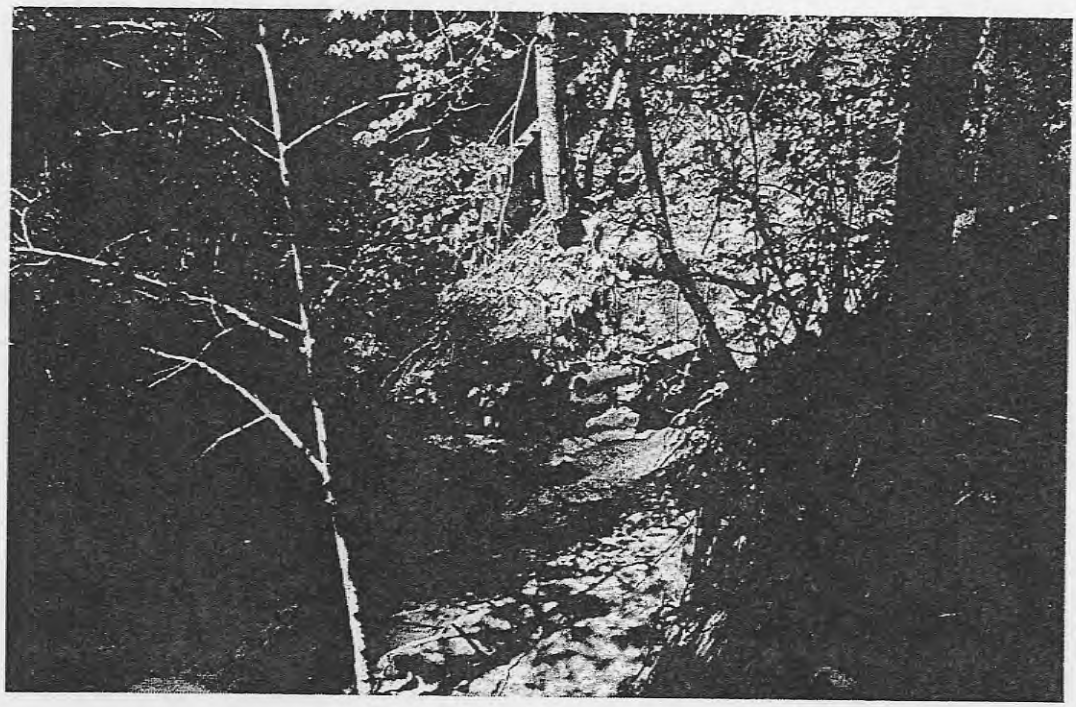


Figure 6: Downstream face of the Upper Reservoir dam built in 1900, showing stone-masonry culvert and outlet pipe *(photograph courtesy of Calif. Dept. of Water Resources).*

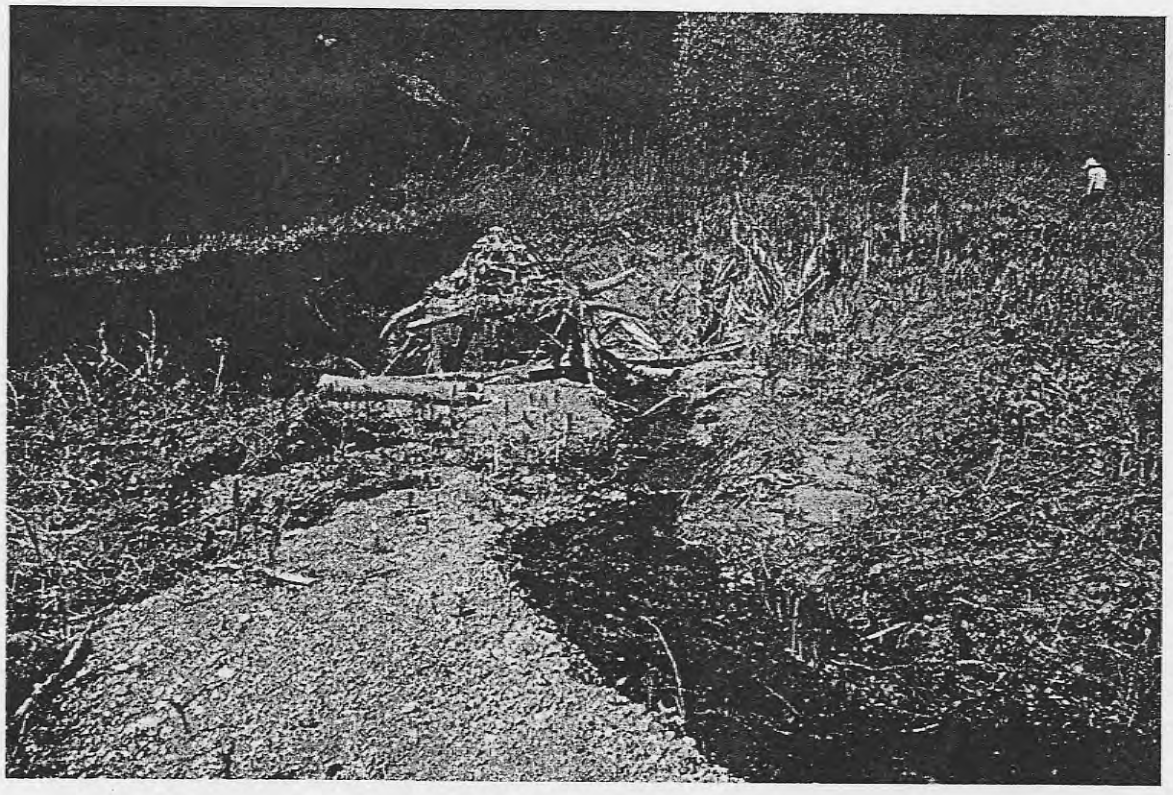


Figure 7: Interior of the Upper Reservoir showing capped vertical stand pipe intake *(Photograph courtesy of Calif. Dept. of Water Resources).*

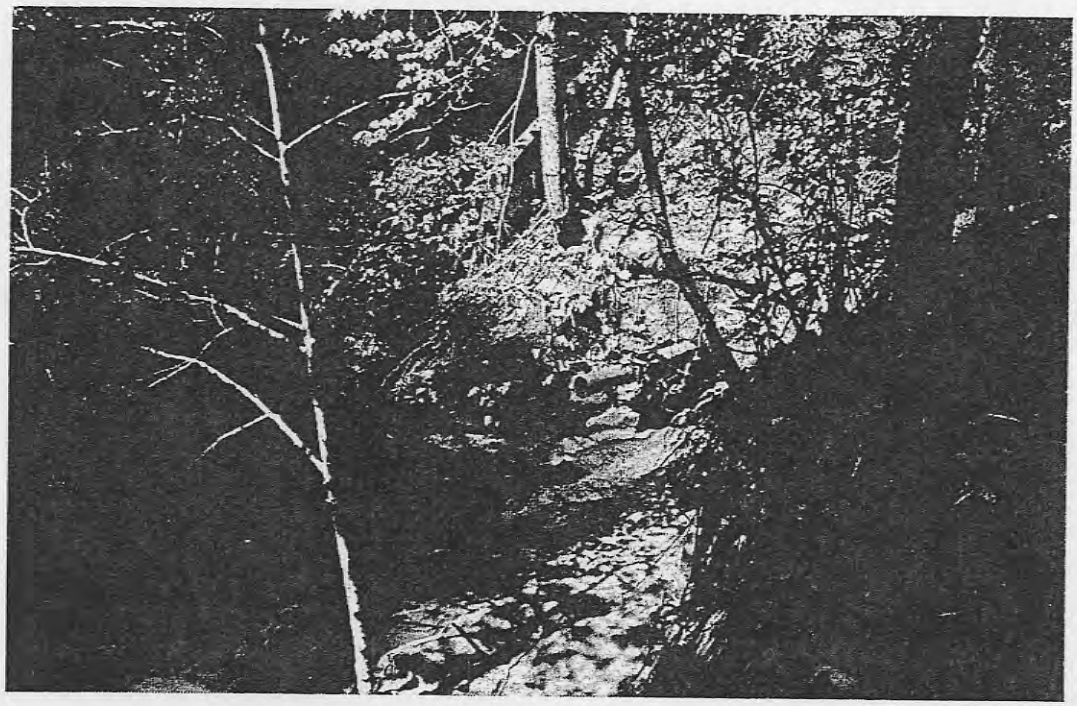


Figure 6: Downstream face of the Upper Reservoir dam built in 1900, showing stone-masonry culvert and outlet pipe *(photograph courtesy of Calif. Dept. of Water Resources).*

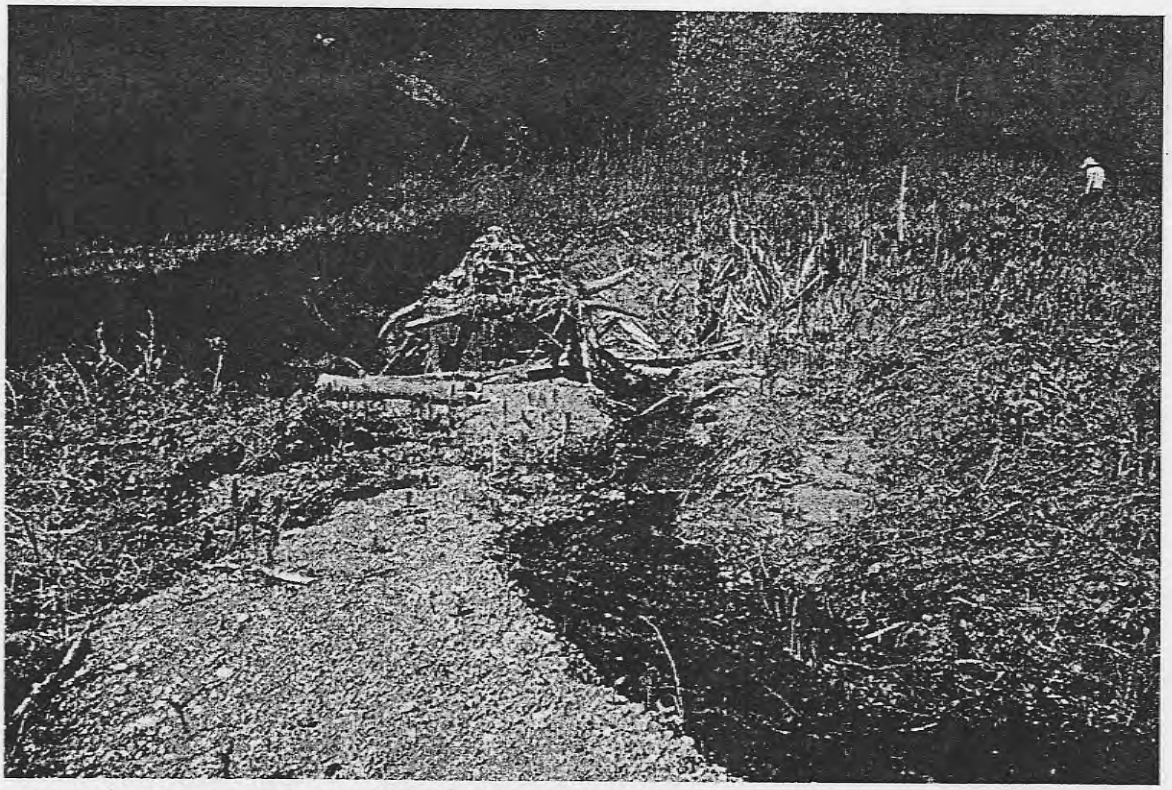


Figure 7: Interior of the Upper Reservoir showing capped vertical stand pipe intake *(Photograph courtesy of Calif. Dept. of Water Resources).*

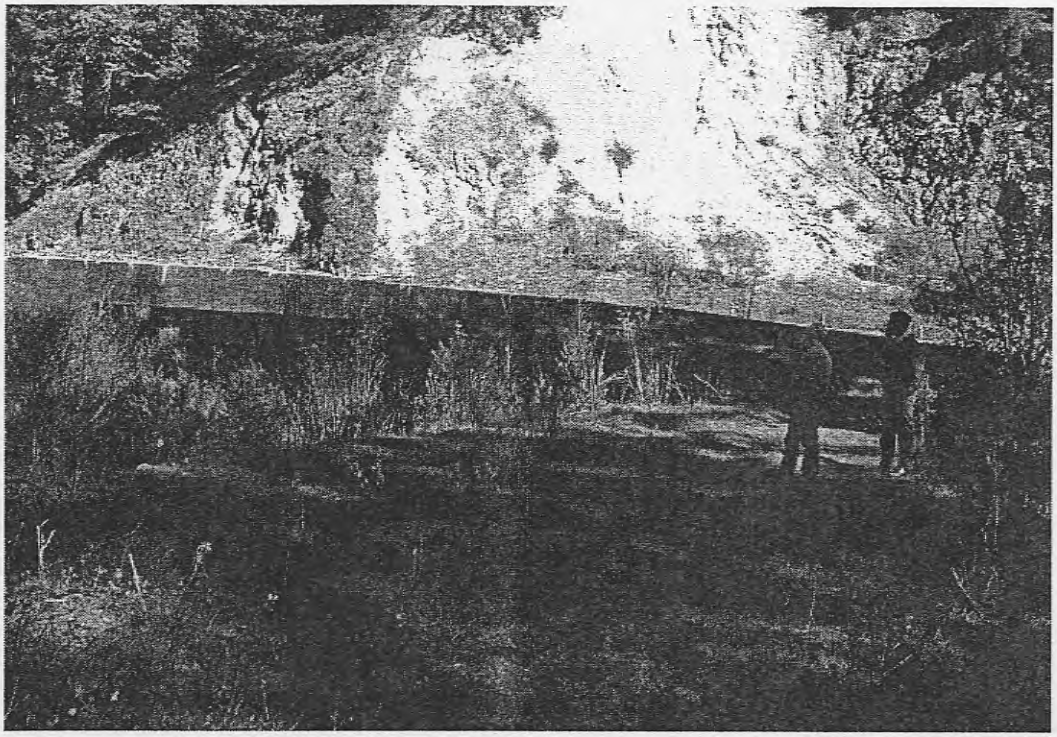


Figure 8: Crest of the Upper Reservoir Dam looking toward Spring Mountain Road *(photograph courtesy of Calif. Dept. of Water Resources).*

York Creek Diversion

The York Creek Diversion structure is located on York Creek approximately one-half mile downstream from the Upper Reservoir along the south side of Spring Mountain Road. It consists of a stone masonry dam built in 1900 through which passes a pipe that is two inches in diameter. Fieldstones, roughly cut, were used with cement in its construction. The dam spans the width of the creek to a height of 12 feet, where the masonry is embedded in the bank on the south side. A 30-inch, corrugated-metal pipe and slide gate valve are located in the center of the dam, 6.8 feet above the creek bed (Figure 9). Three layers of rectangular concrete forms cap the stone masonry span in a stepped pattern. Above the creek bed, rectangular concrete forms that buttress a concrete platform fitted with a 12-inch outlet valve have been built onto the northeast corner of the dam. A concrete apron extends about 12 feet upstream, and a concrete intake box fitted with a welded, 1-inch wire-mesh debris screen has been built onto the northwest corner of the dam (Figure 10). The debris screen is two feet wide and four feet high. The inside of the intake box was fitted with a 16-inch, welded-steel pipe that diverts water via gravity flow from York Creek through an underground pipeline to the Lower Reservoir (Figure 11). This structure has not been altered since its initial construction in 1900.

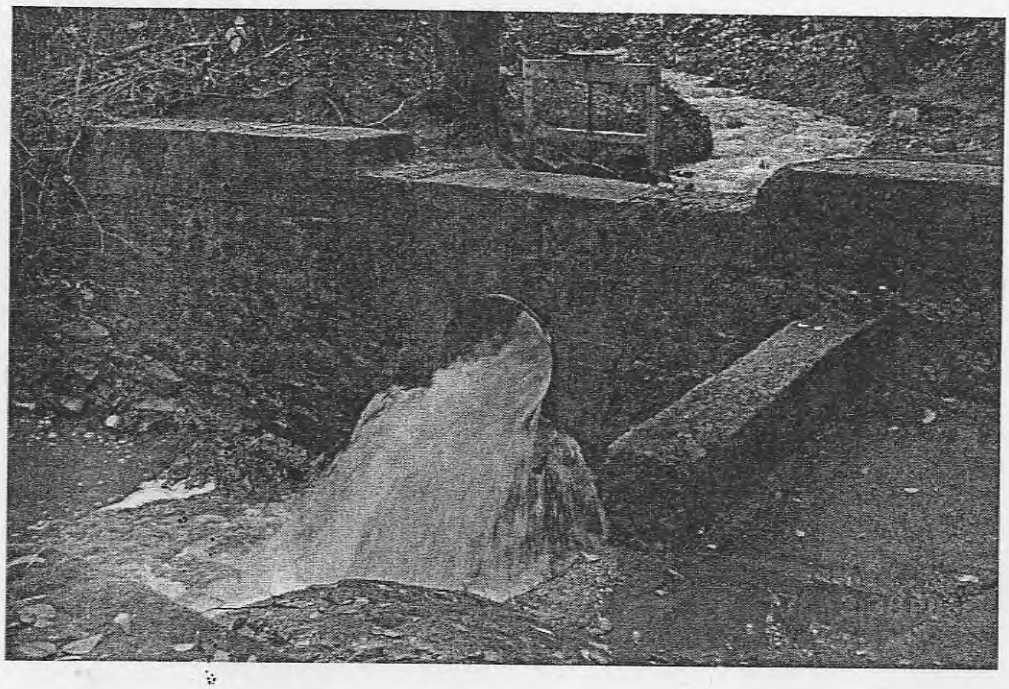


Figure 9: York Creek Diversion dam showing concrete capping, corrugated-metal outlet pipe and concrete buttress platform (*photograph courtesy of Calif. Dept. of Water Resources*).

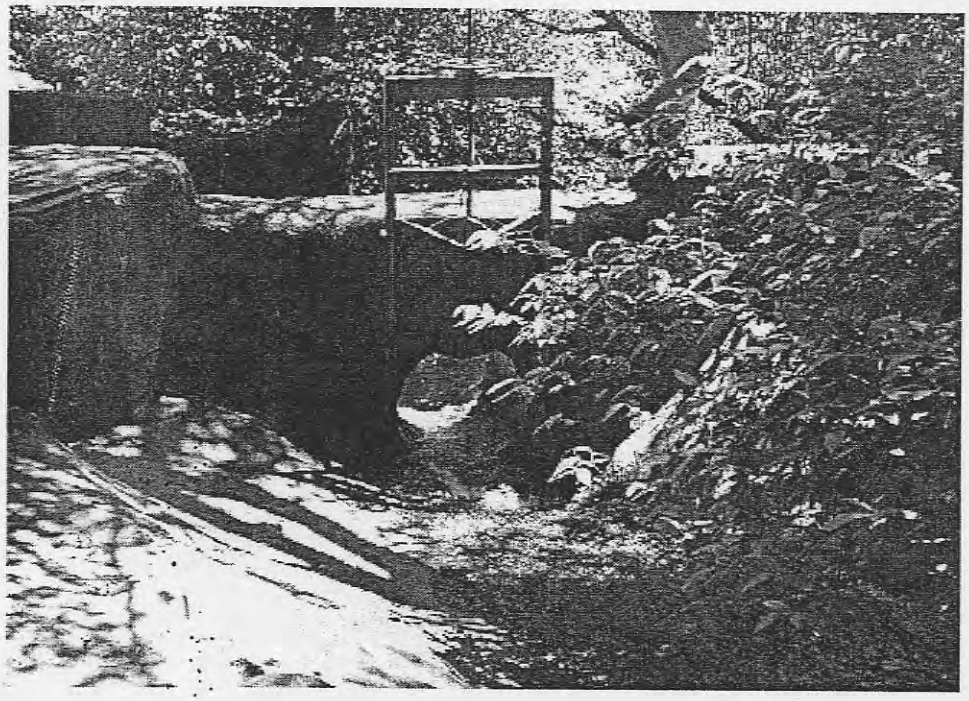


Figure 10: Upstream face of the diversion dam, showing iron slide gate valve and concrete intake box (*photograph courtesy of Calif. Dept. of Water Resources).*

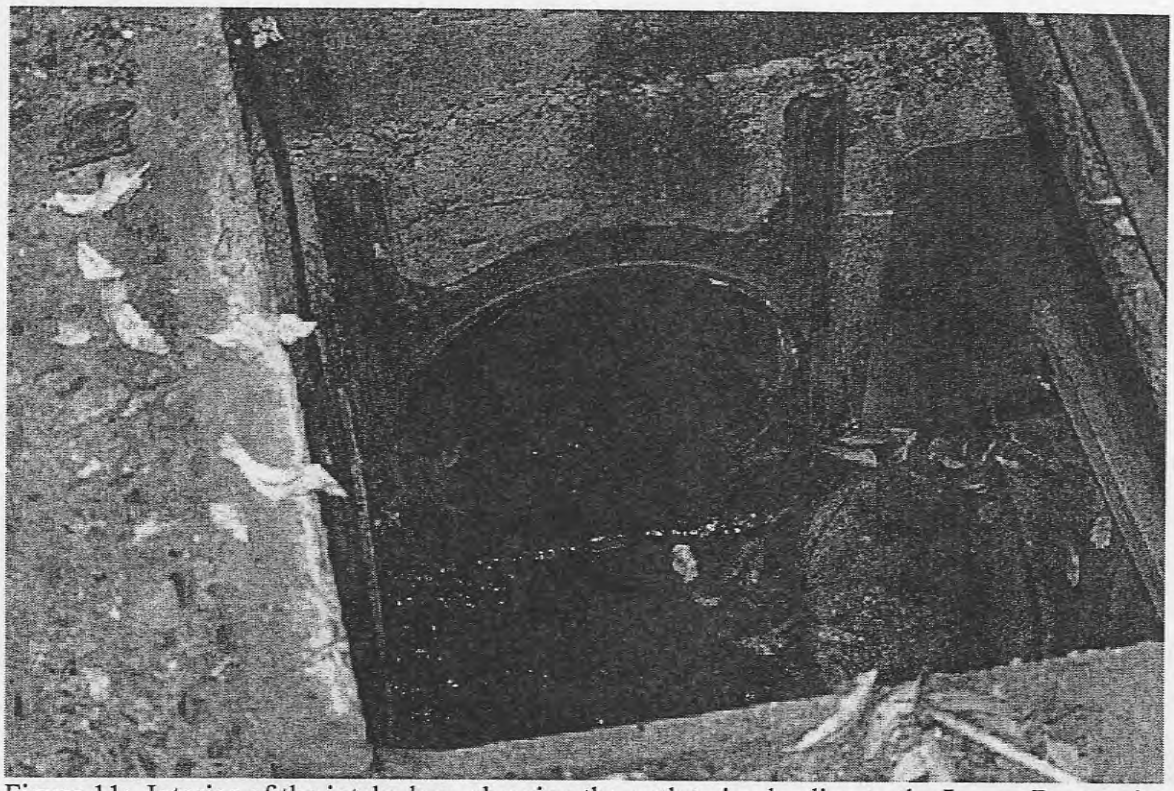


Figure 11: Interior of the intake box, showing the outlet pipe leading to the Lower Reservoir *(photograph courtesy of Calif. Dept. of Water Resources).*

RESOURCE SIGNIFICANCE

The goal of this investigation was to evaluate the eligibility of the resources for inclusion in the National Register of Historic Places, and to assess their status as historical resources for the purposes of CEQA as it pertains to their eligibility for the California Register of Historical Resources (PRC 5024.1). This type of evaluation helps the Corps and the City to meet their obligations with respect to Section 106 of the National Historic Preservation Act of 1966 (16 USC 470), the procedures of the Advisory Council on Historic Preservation (36 CFR 800), and the guidelines of CEQA (Section 15064.5). The structures on York Creek were evaluated according to the criteria of the National Register of Historic Places, using the Secretary of the Interior's Standards and Guidelines for Evaluation detailed in Title 36 CFR part 60.4:

> The quality of significance in American history, architecture, archaeology, engineering, and culture is present in districts, sites, buildings, structures, and objects that possess integrity of location, design, setting, materials, workmanship, feeling, and association, and:

A. that are associated with events that have made a significant contribution to the broad patterns of our history; or

B. that are associated with the lives of persons significant in our past; or

C. that embody the distinctive characteristics of a type, period, or method of construction, or that represent the work of a master, or that possess high artistic values, or that represent a significant and distinguishable entity whose components may lack individual distinction; or

D. that have yielded, or may be likely to yield, information important in prehistory or history.

The Upper Reservoir and Diversion dam were recorded and evaluated as historic components of a larger water-conveyance system that served the city of St. Helena from 1871 through the 1980s, when a new reservoir was constructed. The water system as it was connected to the Upper Reservoir and York Creek Diversion included pipelines from the diversion to the Lower Reservoir, fluoridation treatment equipment; slide gate and spillway, chlorination station, intake and outlet valves, water mains into the city, distribution pipelines, and fire hydrants. Guidelines for evaluating such structures can be found in *Water Conveyance Systems in California*, a document prepared jointly by JRP Historical Consulting Services and the California Department of Transportation (Caltrans) in December 2000. These guidelines were used to inform the historic context statements and evaluations prepared in this report.

The historical significance of the Upper Reservoir and downstream diversion structure can be understood by examining the historical development of typical community water systems in California cities and towns (JRP/Caltrans 2000:70). Water supplies were of utmost importance throughout California, owing to its semi-arid Mediterranean climate that sees very little rainfall for half the year. To solve their water problems, early cities and towns relied on a combination of public and private water systems. Private joint-stock water companies owned dams, reservoirs, and flumes. California's early communities encouraged the organization of local water companies to meet the needs of an established and growing settlement, where water for firefighting, hotels, stables, other business enterprises was a paramount concern.

The St. Helena Upper Reservoir and Diversion appear eligible for listing on the NRHP at a local level of significance under Criterion A for their important contribution to the development of infrastructure specifically water delivery in St. Helena in the 20th century. They have a period of significance of 1900, when they were initially constructed and there have been no substantial modifications to these structures since the date of their original construction. After the Upper Reservoir and Diversion were completed in 1900, the town of St. Helena saw a period of steady growth and architectural development. New commercial structures were built along Main Street and many fine homes were constructed in the popular styles of the early 20th century. Without the presence of the Upper Reservoir and its water storage and delivery system the town would not have been able to sustain continued growth that required adequate water services. In addition to providing crucial water delivery, the Upper Reservoir system was designed to provide sediment-free water that required less filtration upon entering the system mains.

The boundaries of the Upper Reservoir encompass the dam, original concrete spillway located on the south end, and the interior of the reservoir. The long concrete spillway added in 1933 along the north side of the dam and reservoir is a contributing element to the property, whose presence does not detract from the historic integrity of the core structure. With the exception of the disappearance of the wooden flume, the reservoir and dam have

retained integrity of design and materials even though breaches have occasionally taken place. Integrity of location and setting are excellent, and the historic feeling and association of the structure and its surroundings in this steep, forested area are still largely unaffected by modern development.

Overall historic integrity of the Upper Reservoir is high because the majority of its original physical characteristics are intact, as well as the original setting, location, historic feeling and association from 1900. It has remained true to its original design, and almost all of its historic materials are still present in their original locations on the structure. Periodic repair, maintenance, and dredging activities have not altered the historic character of the Upper Reservoir system. Rather, these are acceptable minor alterations that permitted continuing use of the reservoir past the period of significance, a fact that enhances its significance under Criterion A. Similarly, changes on the dam crest or in its vertical measurements have not been so drastic as to obscure its historic appearance. The 1933 addition of the concrete spillway alongside the north side of the reservoir and dam did not destroy the major design characteristics of the reservoir structure or its historic fabric, and therefore does not compromise the ability of the resource to convey is historic significance.

The boundaries of the York Creek Diversion structure encompass the dam that spans the creek, the concrete abutments and apron, and the intake box. As a component of the Upper Reservoir water collection and storage facility, the diversion dam and intake pipes were instrumental for water collection and conveyance. Upon reaching the intake box, the water had deposited most of the sediment from the reservoir on the stones of the creek bed. The water was in a fairly clear and pure state by the time it arrived in the large redwood storage tank near the Lower Reservoir. Although the redwood storage tank no longer exists, the underground pipeline, flume, and valves leading to the original water mains are all still intact. Having excellent historic integrity of original design and materials, along with its original location and setting, the diversion structure clearly conveys its historical importance.

The York Creek Diversion has not sustained any substantial alterations since its construction in 1900, leaving it virtually intact with a high degree of historic integrity of design, materials, and the original workmanship. It remains in its original location, and the absence of surrounding development or alteration of the streambed has preserved the historic setting, feeling, and association. The diversion has high overall integrity and possesses all of the physical characteristics to convey its significance under Criterion A.

The York Creek Water System as a Rural Historic Landscape

The nature of the York Creek dam and diversion warrants their consideration as contributing elements to a rural historic landscape that may be eligible for inclusion in the National Register. National Register guidelines define a rural historic landscape as

> "...a geographical area that historically has been used by people, or shaped or modified by human activity, occupancy, or intervention, and that possesses a significant concentration, linkage, or continuity of areas of land use, vegetation, buildings and structures, roads and waterways, and natural features" (McClelland, Keller, Keller, Melnick n.d.:1-2).

Preliminary observations resulting from this study suggest that the York Creek water system exists as a rural historic landscape. The York Creek water system rural historic landscape consists of:

- a segment of York Creek and its embankment;
- a segment of York Creek canyon and its natural drainage;
- surrounding riparian vegetation and forest
- the Upper Reservoir dam and spillways, and the Diversion;
- the Lower Reservoir, dam, and spillway;
- a segment of Spring Mountain Road and access roads;
- buildings located at the Lower Reservoir and alongside Spring Mountain Road that are associated with the St. Helena water management and conveyance system;
- objects such as pipelines, valves, and mains
- natural hydrology, geology, and topography that make the system possible.

The boundaries that reinforce this cultural landscape as a distinct historical entity consist of the segment of York Creek that enters the Upper Reservoir, then southwesterly across Spring Mountain Road to the western slope above the Lower Reservoir including the low terrain and hills surrounding the Lower Reservoir, then easterly from the chlorination station across Spring Mountain road to the valves alongside the south side of the road, continuing southerly to York Creek and westerly up York Creek to the Diversion structure, thence westerly upstream to the Upper Reservoir dam. The lack of modern development within these boundaries enhances the historic qualities of setting, feeling, location, and association of the historic landscape. Its period of significance is 1878-1900.

Major alterations to the Lower Reservoir have changed its appearance since it was constructed in 1878. If these alterations, however, were completed at least 50 years ago, they may be considered historic alterations that have helped sustain the utility of the property, and therefore do not diminish its historic significance as a contributor. Considering the vital role the St. Helena water storage and conveyance facilities played in the growth of the town in the late 19th century and 20th century, this rural historic landscape appears to be potentially eligible for listing on the National Register.

FINDINGS AND CONCLUSIONS

Two related historical resources were identified as a result of field surveys and archival research: the St. Helena Upper Reservoir and York Creek Diversion. They were recorded on DPR 523 forms that are included in this report as Appendix A and are shown on Sheets 1 and 2 of the APE map (Figure 3). These properties are located within the project APE and were evaluated for their eligibility for listing on the National Register as required by ACHP's Section 106 procedures. Both appear to be eligible for individual listing on the National Register at the local level of significance, and both may be considered historical resources for the purposes of CEQA, and thereby are eligible for listing on the California Register of Historical Resources. Preliminary observations suggest that the resources may also be contributors to a rural historic landscape that appears eligible for the National Register. It is recommended that the York Creek water system landscape be fully documented.

The following properties appear to be eligible for inclusion in the National Register and qualify as historical resources under CEQA:

Name	Location	Community	Map Ref.
Upper Reservoir	south side of Spring Mountain Road, 1.5 miles westerly from corner of Madrona Avenue and Spring Mountain Road.	St. Helena	Sheet 1
York Creek Diversion	south side of Spring Mountain Road, 1 mile westerly from Corner of Madrona Avenue and Spring Mountain Road	St. Helena	Sheet 2

PREPARER'S QUALIFICATIONS

Bright Eastman

Bright Eastman is Staff Architectural Historian for the Anthropological Studies Center at Sonoma State University. She was employed as an Associate Environmental Planner in Architectural History with the California Department of Transportation for two-and-a-half years, in both Districts 04 and 10. She has an M.A. in Cultural Resources Management from Sonoma State University, with cultural geography specialization. She has completed formal training in Section 106 compliance and has four years of qualifying professional experience in evaluating architectural properties for National Register eligibility. Additional qualifying experience includes nine years full- or part-time experience as Project Coordinator for the Anthropological Studies Center, Sonoma State University, and five years as a part-time historical resources consultant in the city of Napa.

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1877 "Water Company Incorporated," 27 July 1877, 2:2, 3:1,
1878a "Water Works Described," 17 May, 1878, 3:2 1878b "Dam Almost Finished," 31 May 1878, 3:1,
1878c "Temporary Water Rates," 2 July 1878, 3:1,
1878d "Hydrants To Be Installed," 2 August 1878, 2:2,
1882b "Improvements Made to Dam," 23 June 1882, 3:2,
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APPENDIX

Historic Resource Inventory Forms

DPR 523

State of California–The Resources Agency
DEPARTMENT OF PARKS AND RECREATION
PRIMARY RECORD

Primary # _____
HRI # _____
Trinomial _____
NRHP Status Code 3S

Other Listings _____
Review Code _____ Reviewer _____ Date _____

 *Resource Name or #: (Assigned by recorder) Upper Reservoir

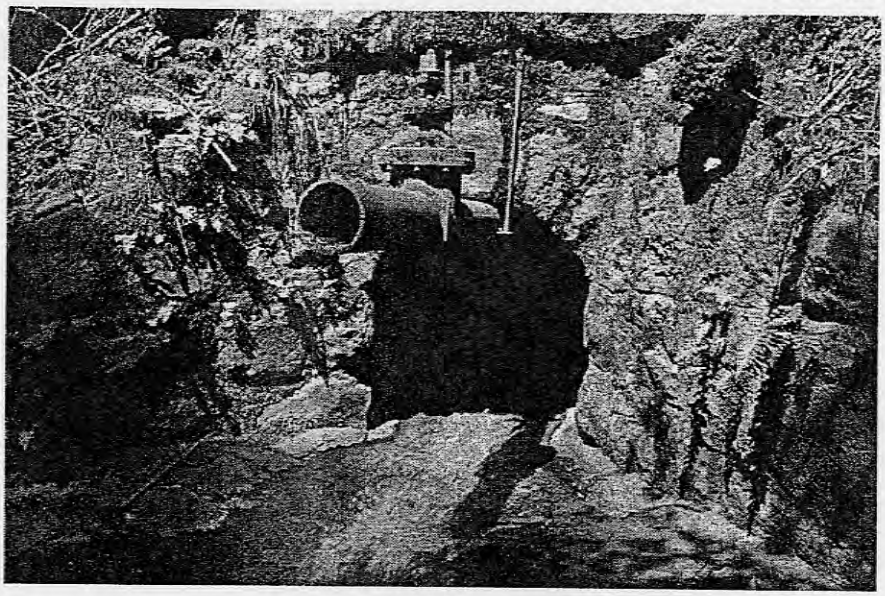
P1. Other Identifier: York Creek Dam/Upper Reservoir Site

*P2. Location: ☐ Not for Publication ☑ Unrestricted *a. County Napa
and
*b. USGS 7.5' Quad Calistoga Date 1958 (pr 1980) T 8N R 6
W, NW¼ of SW ¼ of Sec. 26 ; MDBM
c. Address Spring Mountain Road City St. Helena Zip _____
d. UTM: NEED Zone 10 mN/ _____ mE/ _____
e. Other Locational Data:
1.5 miles westerly on Spring Mountain Road from the corner of Madrona Avenue in St. Helena. Reservoir is situated along the south side of the road.

*P3a. Description:

The City of St. Helena's Upper Reservoir is located on York Creek approximately 1.5 miles upstream from the city along the south side of Spring Mountain Road. The reservoir is abandoned and covers an area approximately 420 feet long and 120 to 180 feet wide. A 6-foot stand-pipe outlet is situated in the bottom of the reservoir, approximately 38 feet behind the dam. Earth excavated in 1900 for the reservoir was used to construct the earthen-filled dam, which is faced on both sides with basaltic fieldstone riprap. The dam rises to a height of 50 feet, with a crest elevation of 620 feet. It is 140 feet long and is about 30 wide on top. The intake and outlet pipes are located at the center of the base of the dam. The remains of the original iron screw gear for the slide gate lies on top of the dam near the southernmost end. A keystone arched stone culvert surrounded by roughly cut stone masonry abutments is located at the center of the outfall on the downstream side of the dam. It measures 3 feet in diameter and contains a 9-inch, riveted pipe and discharge valve. The original concrete spillway extends from the top of the south end of the dam to the creek bed below. See Continuation Sheet.

*P3b. Resource Attributes: (List attributes and codes) HP. 21,Dam, HP.22 Reservoir
*P4. Resources Present: ☑ Building ☐ Structure ☐ Object ☐ Site ☐ District ☐ Element of District ☐ Other (Isolates, etc.)



P5b. Description of Photo: (View, date, accession #)dam culvert and outlet pipe
2002
*P6. Date Constructed/Age and Sources: ☑ Historic, 1900
☐Prehistoric
☐ Both
*P7. Owner and Address:
City of St. Helena
1480 Main Street
St. Helena, CA 94574
*P8. Recorded by:
Bright Eastman
Anthropological Studies Center
Sonoma State University
Rohnert Park, CA 94928
*P9. Date Recorded: 12 September 2002
*P10. Survey Type:Intensive survey
*P11. Report Citation: 2002 Eastman, Bright "Historical Resources Evaluation Report for the Proposed Removal of an Earthen Dam and Diversion Structure on York Creek near the City of St. Helena in Napa County, California". Prepared by the Anthropological Studies Center, Sonoma State University, Rohnert Park, California for the City of St. Helena, St. Helena, California

*Attachments: ☑ Location Map ☐ Sketch Map ☐ Continuation Sheet ☑ Building, Structure, and Object Record☐ Archaeological Record ☐ District Record ☐ Linear Feature Record ☐ Milling Station Record ☐ Rock Art Record☐ Artifact Record ☐ Photograph Record ☐ Other (List) _____

State of California–The Resources Agency
DEPARTMENT OF PARKS AND RECREATION
BUILDING, STRUCTURE, AND OBJECT RECORD

Primary # _____
HRI # _____

***NRHP Status Code** 3S

***Resource Name or #** (Assigned by recorder) Upper Reservoir

B1. Historic Name: Upper Reservoir
B2. Common Name: York Creek Dam
B3. Original Use: Water storage
B4. Present Use: Not in use
***B5. Architectural Style:** earthen dam faced with fieldstone

***B6. Construction History:**
Reservoir and dam constructed in 1900. A redwood flume once carried water from the spillway to the creek. It is no longer present. The giant screw ("worm") gear used to operate the sluice gate on the inner face of the dam. It has been removed from its original location and placed on top of the dam.

***B7. Moved?** ☑ No ☐ Yes ☐ Unknown **Date:** _____ **Original Location:** _____

***B8. Related Features:**
The York Creek Diversion dam that is located approximately one-half mile downstream from the Upper Reservoir

B9a. Architect: J.M. Graham
b. Builder: J.M. Graham

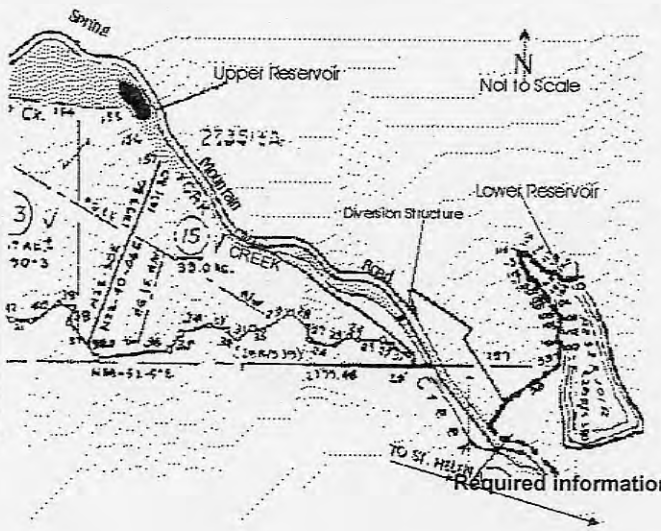
***B10. Significance: Theme** Development of St. Helena **Area** St. Helena, Napa County, California
Period of Significance 1900 **Property Type** Water System **Applicable Criteria** A
The St. Helena Upper Reservoir and Diversion appear eligible for listing on the NRHP at a local level of significance under Criterion A for their important contribution to the development of infrastructure, specifically water delivery, in St. Helena in the 20th century. They have a period of significance of 1900, when they were initially constructed and there have been no substantial modifications to these structures since the date of their original construction. After the Upper Reservoir and diversion were completed in 1900, the town of St. Helena saw a period of steady growth and architectural development. New commercial structures were built along Main Street and many fine homes were constructed in the popular styles of the early 20th century. The architectural heritage of the City of St. Helena today contains not only commercial and vernacular rural architecture from the 19th century, but also well-executed examples of early 20th century revival styles and many fine Craftsman style homes of the 1900s through the 1930s. Without the presence of the Upper Reservoir and its water storage and delivery system the town would not have been able to sustain continued growth that required adequate water services. See Continuation Sheet.

B11. Additional Resource Attributes: (List attributes and codes) _____

***B12. References:** Napa County Official Records Volume 135: 281-283, 22 May 1922. *St. Helena Star* 22/6/ 1900, 3:2. *Napa Daily Journal* 9/11/ 1899, 3:2, Map *of St. Helena, California.* Prepared by Kenneth J. Miller, n.p., St. Helena, California, 1924. "Report on Acquisition of the Properties of St. Helena Water Company By Town of St. Helena." Report prepared by F. Emerson Hoar, Consulting Engineer, for the Board of Trustees and the Chamber of Commerce, St. Helena, California, 28 March 1922, San Francisco, California, *History of Solano and Napa Counties, California*, Gregory, Tom Historic Record Company, Los Angeles, California 1912, Loeber, Ivy M. 1955 History *of St. Helena – A Century of Progress*, Napa County Historical Association, Napa, California. Brownell, Robert, 2002 (Water Foreman for the City of St. Helena) Personal communication with Bright Eastman on 27 September 2002 in St. Helena, California.

B13. Remarks:
***B14. Evaluator:**
Bright Eastman
Anthropological Studies Center
Sonoma State University
Rohnert Park, CA 94928
***Date of Evaluation:** 7 October 2002

(This space reserved for official comments)



*Required information

P3a. Description, continued

A second spillway 270 feet in length was constructed adjacent to the north end of the dam alongside Spring Mountain Road. It is concrete-lined and flows downstream to a collection pond at the north side of the creek below the dam outlet The reservoir and dam have not been reconstructed or altered in a major way since initial construction in 1900. Routine repair and maintenance of existing parts were performed on the structure in the 1970s. Modifications to bring the dam into compliance with safety standards were completed in 1985, but these alterations did not change the historic appearance of the structure, or negatively affect its original setting, feeling, or association with the original design (DWR 2003).

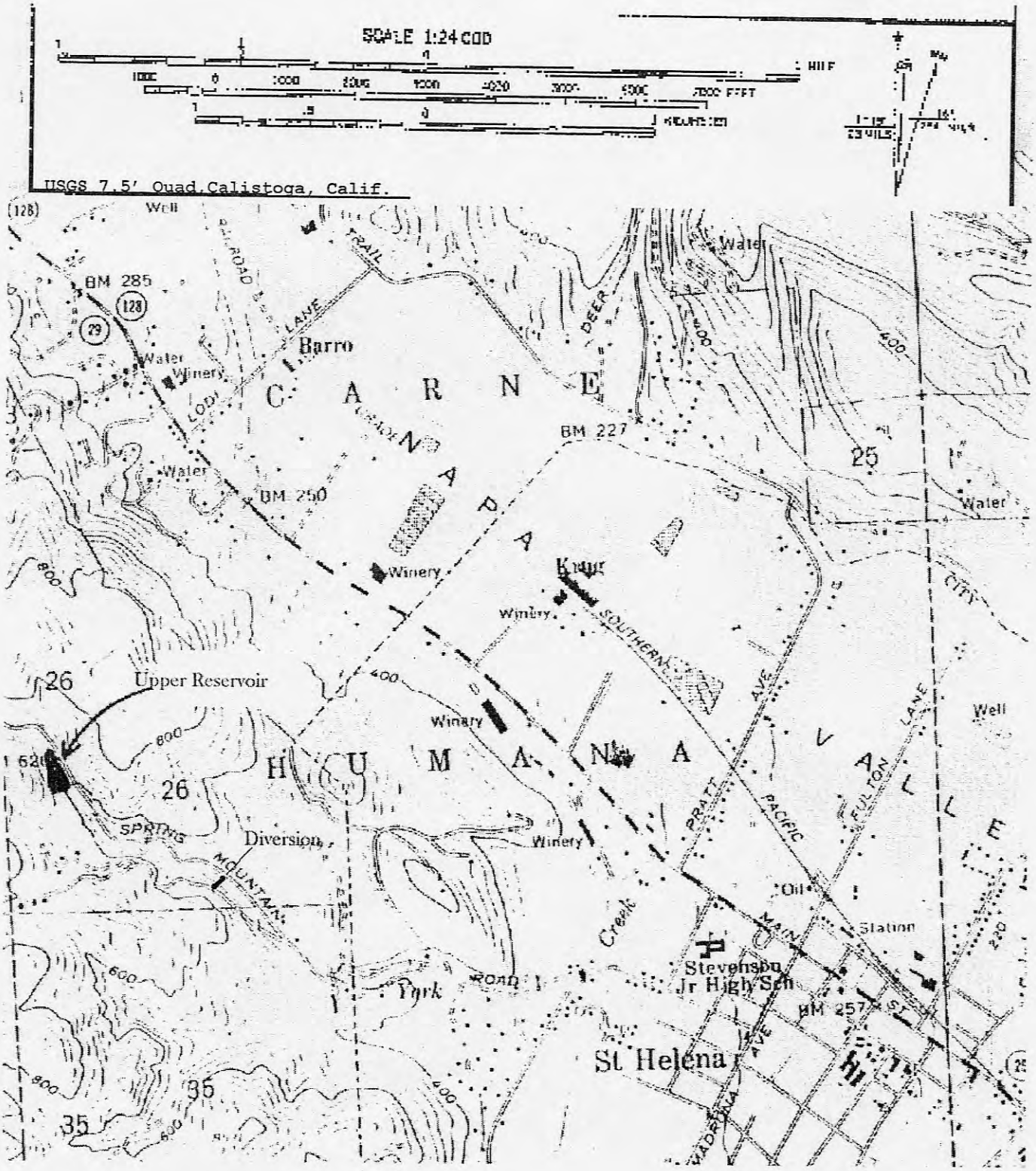
B10. Significance, continued

The boundaries for the Upper Reservoir includes the dam, original concrete spillway located on the south end, and the interior of the reservoir. The long concrete spillway added along the north side of the dam and reservoir is a contributing element, whose presence does not detract from the historic integrity of the core structure. With the exception of the disappearance of the wooden flume, the reservoir and dam have historic integrity of design and materials even though breaches have occasionally taken place. Integrity of location and setting are excellent, and the historic feeling and association of the structure and its surroundings is still largely unaffected in this steep, forested area.

*Resource Name or #: Upper Reservoir

*Recorded by: Bright Eastman
DPR 523L (1/95)

*Date: 12 September 2002 ☒ Continuation ☐ Update



State of California–The Resources Agency
DEPARTMENT OF PARKS AND RECREATION
PRIMARY RECORD

Primary # _____
HRI # _____
Trinomial _____
NRHP Status Code 3S

Other Listings _____
Review Code _____ Reviewer _____ Date _____

 *Resource Name or #: (Assigned by recorder) York Creek Diversion Structure

P1. Other Identifier: York Creek Diversion

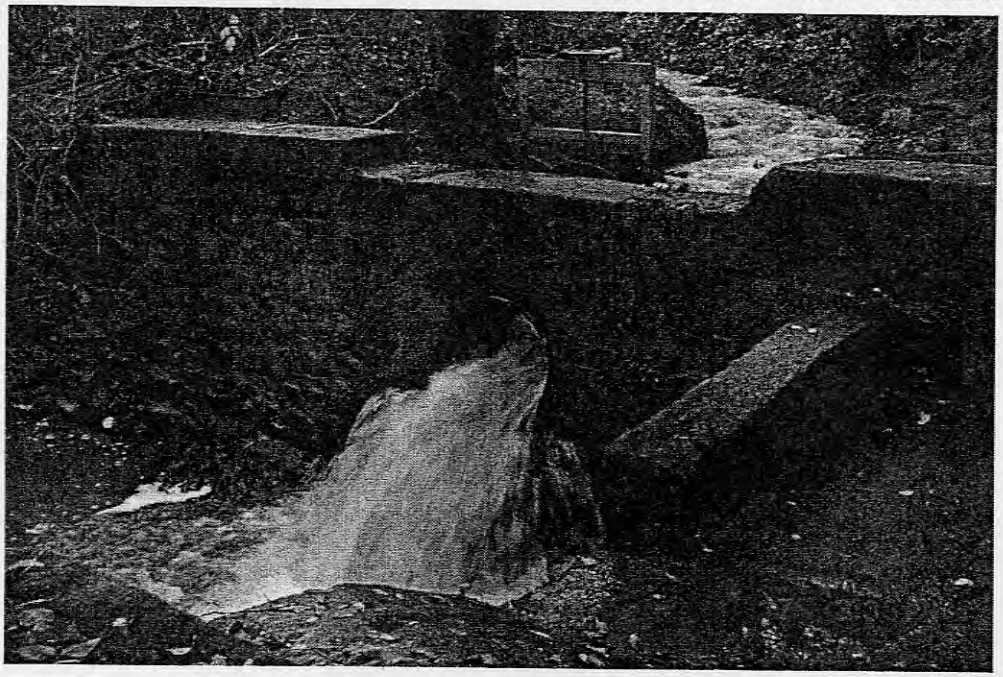
*P2. Location: ☐ Not for Publication ☑ Unrestricted *a. County Napa
and (P2b and P2c or P2d. Attach a Location Map as necessary.)
*b. USGS 7.5' Quad St. Helena Date 1960 (pr 1980) T 8N R 6W, SW¼ of SE ¼ of Sec. 26 ; MDBM
c. Address Spring Mountain Road City St. Helena Zip _____
d. UTM: YOU NEED - Zone 10 mN/ _____ mE/ _____
e. Other Locational Data:
Situated on York Creek one mile westerly on Spring Mountain Road from the corner of Madrona Avenue in St. Helena.

*P3a. Description: (Describe resource and its major elements. Include design, materials, condition, alterations, size, setting, and boundaries).

The York Creek Diversion structure is located on York Creek approximately one mile upstream from the city of St. Helena along the south side of Spring Mountain Road. It consists of a stone masonry dam built in 1900 through which passes a cast-iron pipe two inches in diameter. Fieldstones, roughly cut, were used with cement in its construction. The dam spans the width of the creek to a height of 12 feet, where the masonry is embedded in the bank on the south side. A 30-inch, corrugated-metal screw pipe and slide gate are located in the center of the dam, 6.8 feet above the creek bed. Three layers of rectangular concrete forms cap the stone masonry span in a stepped pattern. Above the creek bed, rectangular concrete forms that buttress a concrete platform fitted with a 12-inch outlet valve have been built onto the northeast corner of the dam. A concrete apron extends about 12 feet upstream, and a concrete intake box fitted with a welded, 1-inch wire-mesh debris screen has been built onto the northwest corner of the dam . The debris screen is two feet high and four feet wide. The inside of the intake box was fitted with a 16-inch, welded-steel pipe that diverted water via gravity flow from York Creek through an underground pipeline to the Lower Reservoir. This structure has not been altered since its initial construction in 1900. It appears exactly as it did in 1900.

*P3b. Resource Attributes: (List attributes and codes) HP. 21Dam

*P4. Resources Present: Building x☐ Structure ☐ Object ☐ Site ☐ District ☐ Element of District ☐ Other (Isolates, etc.)



P5b. Description of Photo: (View, date, accession #) dam culvert and outlet pipe,2002

*P6. Date Constructed/Age and Sources: ☑ Historic, 1900
☐Prehistoric
☐ Both

*P7. Owner and Address:
City of St. Helena
1480 Main Street
St. Helena, CA 94574

*P8. Recorded by:
Bright Eastman
Anthropological Studies Center
Sonoma State University
Rohnert Park, CA 94928

*P9. Date Recorded: 12 September 2002

*P10. Survey Type: Intensive survey

*P11. Report Citation: 2002, Bright "Historical Resources Evaluation Report for the Proposed Removal of an Earthen Dam and Diversion Structure on York Creek near the City of St. Helena in Napa County, California". Prepared by the Anthropological Studies Center, Sonoma State University, Rohnert Park, California for the City of St. Helena, St. Helena, California

*Attachments: ☑ Location Map ☐ Sketch Map ☐ Continuation Sheet ☑ Building, Structure, and Object Record
☐ Archaeological Record ☐ District Record ☐ Linear Feature Record ☐ Milling Station Record ☐ Rock Art Record
☐ Artifact Record ☐ Photograph Record ☐ Other (List) _____

State of California–The Resources Agency
DEPARTMENT OF PARKS AND RECREATION
BUILDING, STRUCTURE, AND OBJECT RECORD

Primary # _____
HRI # _____

***NRHP Status Code** 3S

***Resource Name or #** (Assigned by recorder) York Creek Diversion

B1. Historic Name: Upper Reservoir and Diversion Dam
B2. Common Name: York Creek Diversion
B3. Original Use: Water storage and diversion
B4. Present Use: Not in use
***B5. Architectural Style:** field stone masonry and concrete forms
***B6. Construction History:**
The diversion dam and intake box were built in 1900 as part of the Upper Reservoir system.

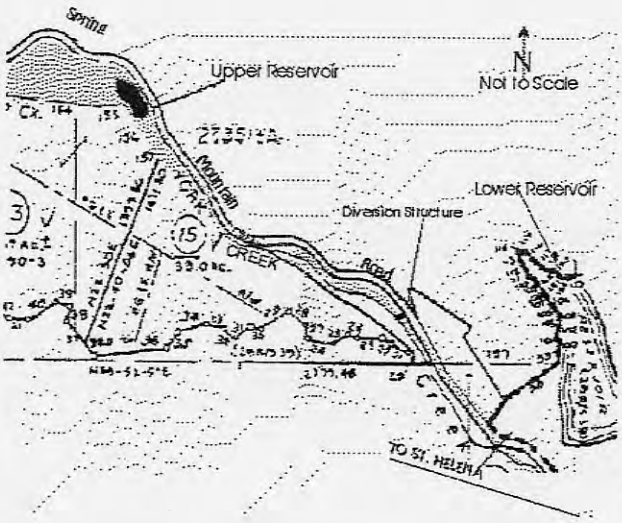
***B7. Moved?** ☑ No ☐ Yes ☐ Unknown **Date:** _____ **Original Location:** _____
***B8. Related Features:**
The St. Helena Upper Reservoir located about one-half mile upstream on York Creek

B9a. Architect: J.M. Graham
b. Builder: J.M. Graham
***B10. Significance: Theme** Development of of St. Helena **Area** St. Helena, Napa County, California
Period of Significance 1900 **Property Type** Water System **Applicable Criteria** A
The St. Helena Upper Reservoir and Diversion appear eligible for listing on the NRHP at a local level of significance under Criterion A for their important contribution to the development of infrastructure, specifically water delivery, in St. Helena in the 20th century. They have a period of significance of 1900, when they were initially constructed and there have been no substantial modifications to these structures since the date of their original construction. Without the presence of the Upper Reservoir and Diversion, the town would not have been able to sustain continued growth that required adequate water services. In addition to providing crucial water delivery, the Diversion was designed to provide sediment-free water that required less filtration upon entering the system mains. The boundaries for the York Creek Diversion structure include the dam that spans the creek, the concrete abutments and apron, and the intake box. As a component of the Upper Reservoir water collection and storage facility, the diversion dam and intake pipes were instrumental for water collection and conveyance. Upon reaching the intake box, the water had deposited most of the sediment from the reservoir on the stones of the creek bed. It arrived in a fairly clear and pure state by the time it arrived in the large redwood storage tank near the Lower Reservoir. The only component of this structure that no longer exists is the redwood storage tank, but the underground pipeline, flume, and valves leading to the original water mains are all still intact even though the diversion is now abandoned. Having excellent historic integrity of original design and materials, along with its original location and setting, the diversion structure clearly conveys its historical importance as a water diversion.

B11. Additional Resource Attributes: (List attributes and codes) _____
***B12. References:** Napa County Official Records Volume 135: 281-283, 22 May 1922. *St. Helena Star* 22/6/ 1900, 3:2. *Napa Daily Journal* 9/11/ 1899, 3:2, Map *of St. Helena, California.* Prepared by Kenneth J. Miller, n.p., St. Helena, California, 1924. "Report on Acquisition of the Properties of St. Helena Water Company By Town of St. Helena." Report prepared by F. Emerson Hoar, Consulting Engineer, for the Board of Trustees and the Chamber of Commerce, St. Helena, California, 28 March 1922, San Francisco, California, *History of Solano and Napa Counties, California*, Gregory, Tom Historic Record Company, Los Angeles, California 1912, Loeber, Ivy M. 1955 History *of St. Helena – A Century of Progress*, Napa County Historical Association, Napa, California. Brownell, Robert, 2002 (Water Foreman for the City of St. Helena) Personal communication with Bright Eastman on 27 September 2002 in St. Helena, California.

B13. Remarks:
***B14. Evaluator:**
Bright Eastman
Anthropological Studies Center
Sonoma State University
Rohnert Park, CA 94928
***Date of Evaluation:** 7 October 2002

(This space reserved for official comments)



n

Description, continued

A 270-foot long concrete spillway was constructed along the north side of the dam and reservoir next to the road in 1932. Concrete was also poured in front of the culvert and at the outfall of the 1932 spillway to create a pond. With the exception of the concrete work added in 1932, the dam and reservoir remain essentially unaltered since the last phase of major improvements that were made in 1883. Natural impacts of occasional breaching are visible, but the historic measurements and materials remain virtually intact.

Signivicance:, continued

The original reservoir was excavated twice since it was first built, in 1882 and 1883, to expand its holding capacity. The earthen dam was also widened, heightened, and lengthened during these improvements to withstand the force of increased cubic water footage. These modifications occurred within the period of significance and were essential for its continued service until the turn of the century.

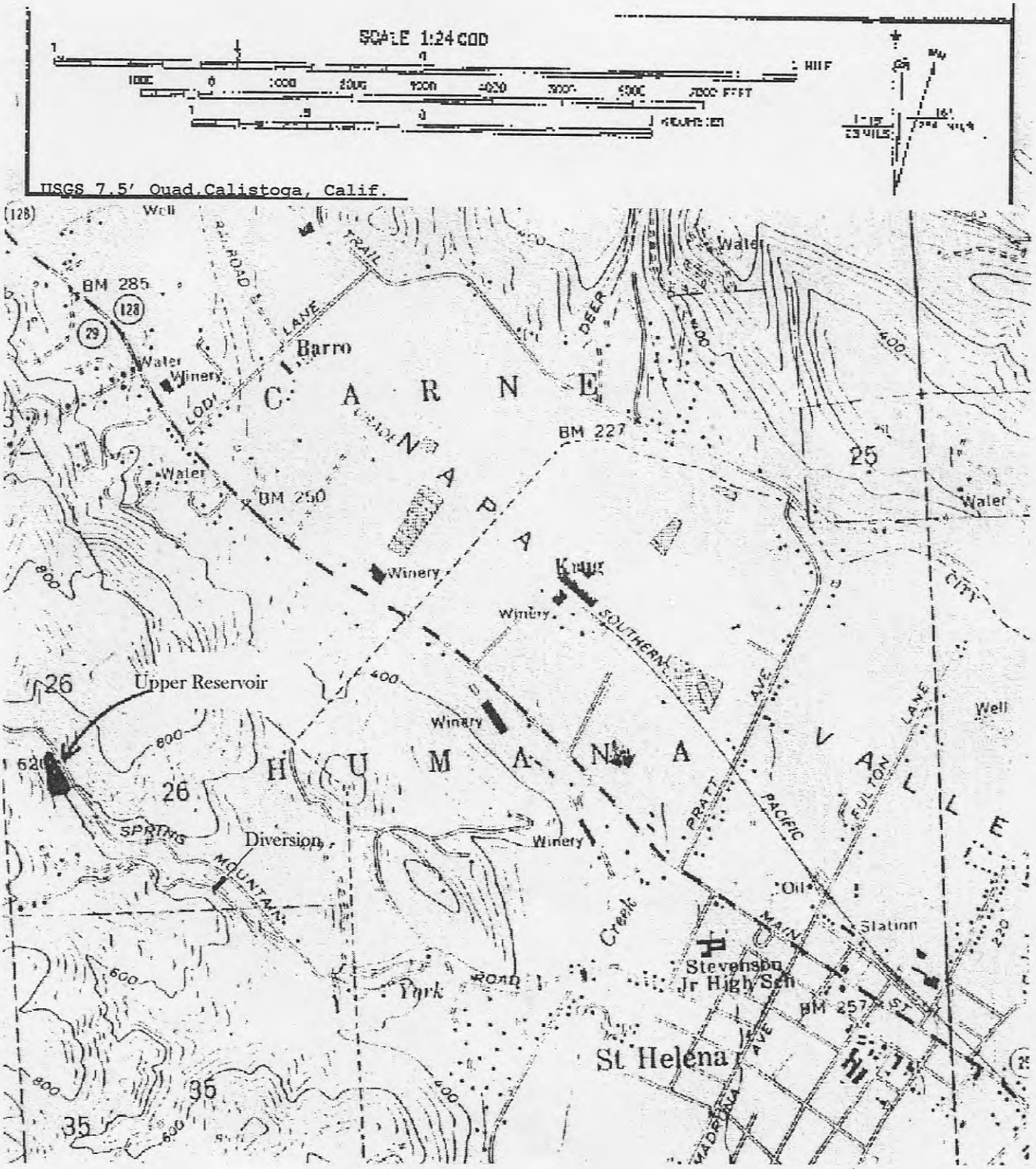
The boundaries for the eligible resource include the dam, original concrete spillway located on the south end, and the interior of the reservoir. The long concrete spillway added alongside the north side of the dam and reservoir in 1932 is a non-contributing element and its presence does not detract from the historic integrity of the core structure. The reservoir and dam have historic integrity of materials and visible phases of nineteenth century construction, even though breaches have occasionally taken place. Integrity of location and setting are excellent, and the historic feeling and association of the structure and its surroundings is still largely unaffected in this steep, forested area.

*Resource Name or #: York Creek Diversion

*Recorded by: Bright Eastman

*Date: 12 September 2002 ☒ Continuation ☐ Update

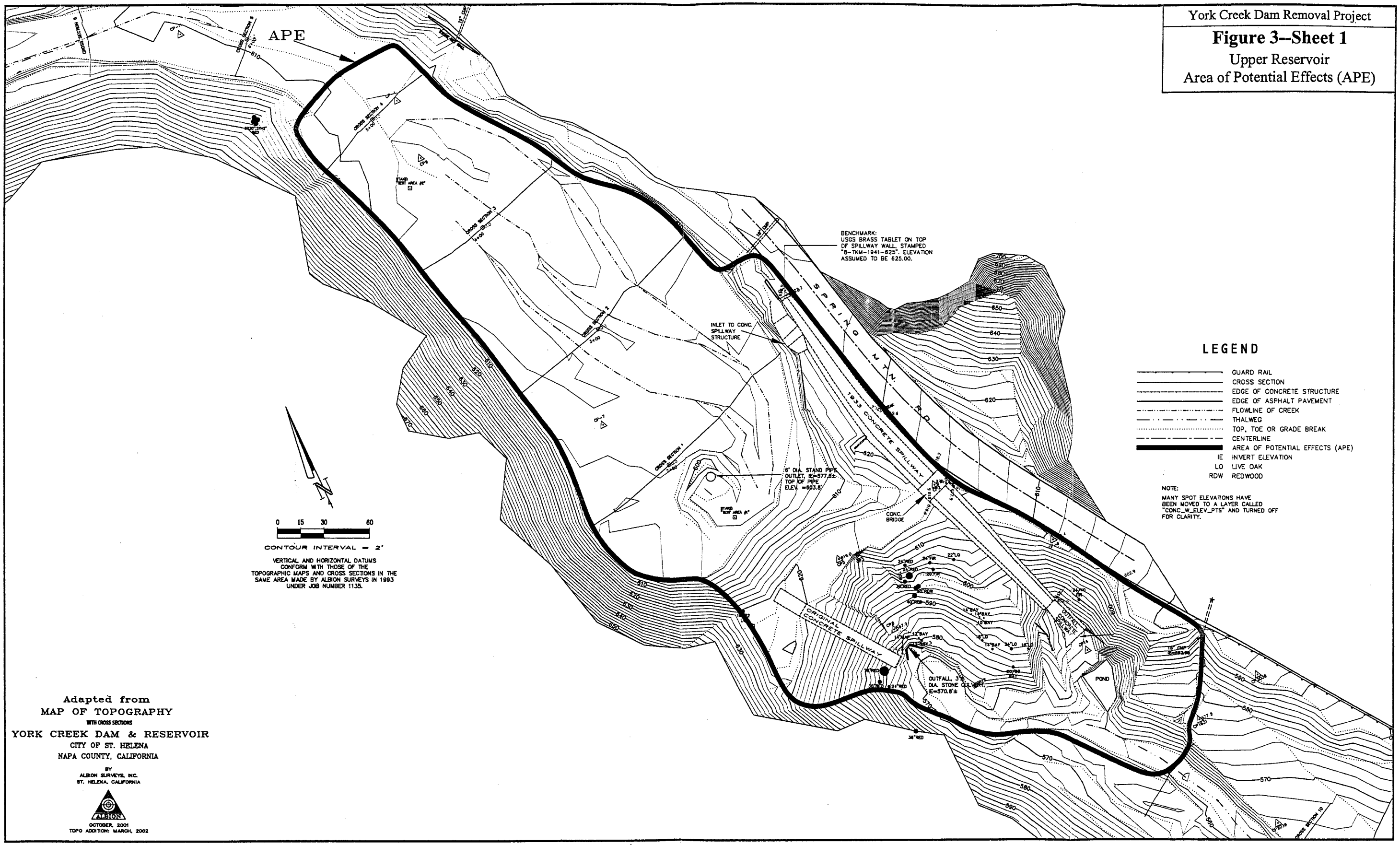
DPR 523L (1/95)



York Creek Dam Removal Project

Figure 3--Sheet 1

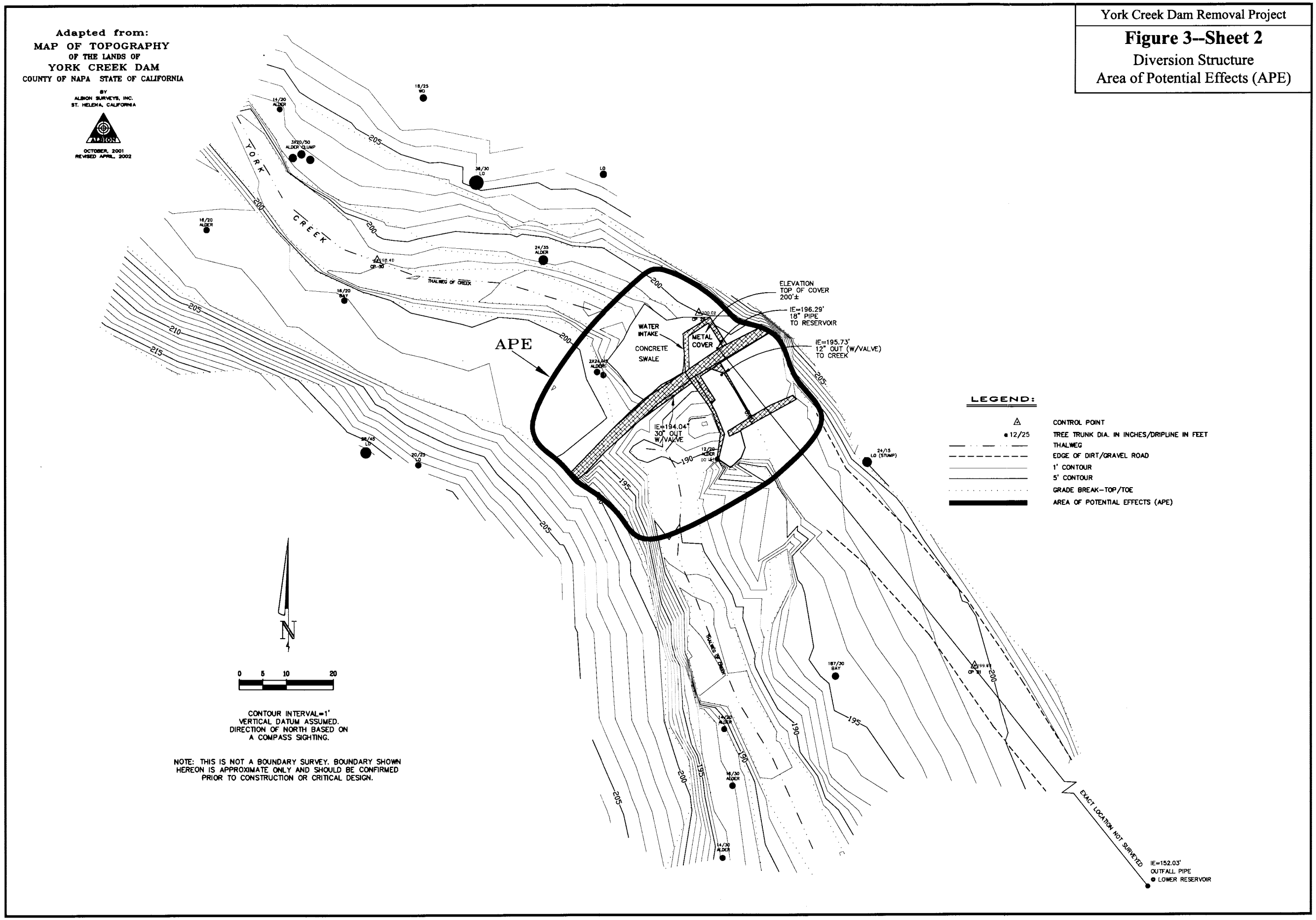
Upper Reservoir
Area of Potential Effects (APE)



York Creek Dam Removal Project

Figure 3--Sheet 2

Diversion Structure
Area of Potential Effects (APE)



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TCJ.

UPPER YORK CREEK DAM REMOVAL PROJECT
SECTION 206: AQUATIC ECOSYSTEM RESTORATION
WITHOUT PROJECT HYDROLOGY CERTIFICATION

Lisa Romanoski of the Water Resources Section of the San Francisco Army Engineer District completed the hydrologic investigation for the proposed Upper York Creek Dam Removal Project. The project site is in St. Helena, CA. All quality control activities, appropriate to the level of risk and complexity inherent in the project, have been completed. Compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This included review of assumptions; methods, procedures and material used in analyses; the appropriateness of data used; and reasonableness of the results. Review of this work was also performed by Bill Firth, a senior level engineer in the Water Resources Section.

S.T. Su — 3-10-05
Chief, Water Resources Section — Date

Herb Cheong — 3/15/05
Chief, Engineering Branch — Date

UPPER YORK CREEK DAM REMOVAL PROJECT
SECTION 206: AQUATIC ECOSYSTEM RESTORATION
BASIN HYDROLOGY ASSESSMENT

MARCH 2005

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UPPER YORK CREEK DAM REMOVAL PROJECT
SECTION 206: AQUATIC ECOSYSTEM RESTORATION
BASIN HYDROLOGY ASSESSMENT

1. PURPOSE

The purpose of this report is to present the results along with the method of analysis of a hydrologic investigation of the Upper York Creek watershed above the city of St. Helena in Napa County. A channel capacity assessment of approximately 3000 feet of Upper York Creek was also done. Results include unit hydrographs for the upper reservoir basin and the lower basin, a storage-elevation curve for the dam, and peak discharge data for the 2-, 10-, 25-, 50-, and 100-year return periods at both the dam and at the confluence of York Creek with the Napa River. See Figure 1 in Appendix A for a location map. The results of this study will serve to help define without project conditions for the Upper York Creek Dam Removal Project.

2. PROJECT DESCRIPTION

The proposed Upper York Creek Dam Removal Project is a restoration project that involves removal of an earthen dam and accumulated sediment from Upper York Creek 1.25 miles northwest of the city of St. Helena. The dam is approximately 50 feet high and has an existing capacity of 20 acre-feet. The base of the dam is at an elevation of 570 feet, and its crest is at an elevation of 620 feet. According to the California Department of Water Resources (DWR), the depth of sediment accumulation behind the dam is approximately 25 feet and extends 600 feet upstream (DWR 2002).

The reservoir, referred to as the Upper Reservoir, has both a concrete spillway that follows Spring Mountain Road adjacent to the project site and a drain pipe spillway behind the dam crest. The concrete spillway is approximately 255 feet long and varies in width along its length. The drain pipe spillway consists of a 6-foot diameter steel riser pipe with a trash rack on top. It extends down 26 feet and connects with a stone masonry tunnel at elevation 577.6 feet. The stone masonry tunnel is 175 feet long, 3 feet in diameter, and has an outlet at the base of the dam at elevation 570.8 feet (DWR 2002).

3. BASIN DESCRIPTION

The Upper York Creek watershed originates on the western side of the Napa Valley watershed at an elevation of approximately 2200 feet. The creek flows in an easterly direction through a narrow canyon before joining the Napa River northeast of St. Helena at an elevation of approximately 225 feet (DWR 2002).

For this study, the watershed was divided into three areas. The area encompassing the ridge tops of the watershed to the top of the dam is 2.48 square miles. The area below the dam to the diversion structure is .35 square miles. The area below the diversion structure to the confluence of York Creek with the Napa River is 2.18 square miles.

Land use in the Upper York Creek watershed consists mostly of forested open space, agriculture, and residential. Vineyards are located within the watershed. Two large wineries are located on the valley floor near the confluence of York Creek and the Napa River. Residential areas within the city limits of St. Helena occur primarily between Highway 12 (river mile 1.0 of York Creek) and approximately river mile 1.75 of York Creek (DWR 2002).

As mentioned previously, Spring Mountain Road is a two-lane county road adjacent to Upper York Creek. It parallels the creek for nearly 2.5 miles and crosses the creek via three bridges. Several culverts carry water from unnamed tributaries that run under Spring Mountain Road and into Upper York Creek. Three of the culverts empty into Upper Reservoir (DWR 2002).

According to the Federal Emergency Management Agency flood insurance study for the city of St. Helena, the only source of flooding affecting St. Helena is the Napa River. Heavy rainfall occurs predominantly during December, January, and February. The FEMA study suggests that streamflow of flood-producing magnitude is the result of storms causing precipitation over the entire river basin for periods in excess of 12 hours. ?

4. AVAILABLE DATA

No design drawings exist for the dam. A topographical survey of the dam and the surrounding area was done in 2001 and 2002 by Albion Surveys. This survey was used as input to various models throughout the course of this study. Another survey done by Albion of the diversion structure downstream of the dam was used to get a cross section for the channel capacity assessment.

There is no stream gage data for Upper York Creek. Flood history is relatively unknown, although the dam was overtopped in 1995. No details of the overtopping event exist. No documentation exists on the pre-existing channel slope prior to dam construction.

There have been four documented silt discharges from the dam into Upper York Creek that have caused kills of fish and other aquatic organisms. The most recent silt discharge occurred in July 1992 during routine maintenance of the reservoir outlet structure (DWR 2002). The amount of sediment released during the discharges is unknown.

The reservoir has been periodically dredged, with one of the last dredging efforts completed in 1993. Following this dredging episode, approximately 9500 yd^3 of sediment has re-deposited in the reservoir (DWR 2002).

5. STUDY METHODOLOGY

The U.S. Army Corps of Engineers' hydrologic program, Hydrologic Modeling System (HMS), was used to simulate the precipitation-runoff process for the Upper York Creek

watershed. For this study, the watershed was divided into three sub-basins for the HMS model in order to look at peak discharge rates for York Creek in certain parts of the watershed. The three sub-basins are modeled are the three areas described in Section 3. Peak flow rates were determined for the sub-basin above the dam for various return periods.

Peak flow rates from the HMS model were then compared to other data, either from other models or data for similar basins. Since DWR used the TR-55 model to determine peak flow rates for this project, TR-55 was run to determine if the DWR peak discharge rates could be reproduced using HMS input values and assumptions.

6. HMS MODEL DEVELOPMENT

6.1. Watershed Characteristics

The watershed characteristics used to develop 0.25 hour unit hydrographs for both sub-basins are listed below.

Table 1. Watershed Characteristics for Upper York Creek Basin

Characteristics	Basin 1	Basin 2	Basin 3
Area (mi^2)	2.48	0.35	2.18
N	0.09	0.09	0.09
L (mi)	3.13	0.38	2
L_{ca}	1.5	0.28	1.8
Slope (ft/mi)	480	316	130
Lag (hrs)	1.20	0.31	1.39

Lag = time for the start of runoff to the point where 50 percent of the ultimate discharge reaches the index point, in hours.
N = basin average roughness coefficient, dimensionless (determined from site visit).
L = length of primary watercourse, in miles.
L_{ca} = distance from centroid of the basin to the downstream end of the basin measured along the primary watercourse, in miles.
S = average slope of primary watercourse, in feet per mile.

It should be noted that future land use conditions were not evaluated in this study because Napa County does not plan to further develop the Upper York Creek watershed.

6.2. Precipitation

A hypothetical frequency storm was used as input to the HMS model to determine the greatest runoff amount from the surrounding watershed. Hypothetical frequency storm events are statistically derived and are based on the analysis probability of occurrence in a given area, expressed as an x-percent change of exceedence in any given year. They are balanced with a consistent depth-frequency relationship for each incremental duration throughout the storm event (USACE 2000).

The storm data used in this analysis is based on precipitation data from the St. Helena 4WS gage, Station Number E30764600. Statistical rainfall data compiled by Jim Goodridge, a climatologist with DWR, is listed in Table 2 for this station. This gage falls into the geographic boundary for SCS rainfall distribution IA.

Table 2. Rainfall Depth Duration Frequency (inches) (Goodridge 2003).

Return Period (yrs)	15 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
2	0.27	0.60	0.90	1.17	1.82	2.64	3.73
10	0.38	1.01	1.52	1.97	3.05	4.43	6.26
25	0.54	1.21	1.82	2.37	3.67	5.33	7.52
50	0.61	1.36	2.05	2.66	4.13	5.98	8.45
100	0.67	1.51	2.27	2.94	4.57	6.63	9.37

Since rainfall tends to increase or decrease with changes in elevation, the statistical rainfall data had to be adjusted before they could be used for the York Creek basin area. This was done using an adjustment factor based on the relative difference in normal annual precipitation (NAP) for the gage location and the sub-basin. This assumes that changes in rainfall due to elevation differences can be measured by changes in mean annual precipitation. An example calculation is shown below.

NAP_{basin}/NAP_{gage} x Precip = (37.5/41.73) x 3.73 = 3.36 in

6.3. Losses and Baseflow

The Soil Conservation Service (SCS) method was used to compute losses in the HMS model. Based on the land use characteristics of the watershed, the cover type was assumed to be a combination of forest and vineyard. From the 1978 Soil Survey for Napa County, the soils in the watershed are best represented by the B, C, or D soil groups. The final curve numbers chosen for each return period were also influenced by antecedent moisture conditions (AMC), with the lesser year events having an AMC closer to I and the greater events having an AMC close to II or III. Table 3 lists the curve numbers chosen for use in the model.

Table 3. Curve Numbers (CN) Used in HMS Model

Return Period (yrs)	CN
2	60
10	61
25	65
50	70
100	78

Baseflow for this analysis was assumed to be zero.

6.4. Reservoir Development in HMS

The development of the reservoir, or dam, in the HMS model consisted of entering storage data per elevation. The riser pipe was modeled as an orifice outlet, and the spillway was modeled as a broadcrested spillway. Elevation and storage capacity data, based on the Albion Survey, is listed below in Table 4. Areas were calculated by hand using a planimeter, as well as by using AutoCad and InRoads. (An elevation-storage curve, Figure 2, has been included in Appendix A.)

Table 4. Storage Capacity Per Contour Interval for York Creek Upper Reservoir

Contour Interval (ft)	Storage Capacity (ac-ft)	Storage Capacity (ac-ft) (InRoads Calculation)
600	0.00	-
602	0.06	0.2
604	0.57	0.24
606	1.97	1.07
608	4.07	2.86
610	6.60	5.25
612	9.59	8.17
614	13.06	11.51
616	17.03	-
618	21.48	-
620	26.51	-

Characteristics for the outlet and the spillway are listed in Table 5.

Table 5. Outlet and Spillway Characteristics for HMS Model.

Orifice Outlet	Broadcrested Spillway
Center Elevation = 603.8 ft	Crest Elevation = 609 ft
Cross-sectional Area (ft2) = 28	Crest Width = 6 ft
Discharge Coefficient = 0.7	Discharge Coefficient = 2.8

7. TR-55 MODEL DEVELOPMENT

A TR-55 model was also set up using similar parameters from the HMS modeling effort. These parameters are listed in Table 6.

Manning's n for Upper York Creek was estimated to be 0.030 based on a table of roughness coefficients for various open channel surfaces (Chow et al 1998). The creek was assumed to be a relatively clean, straight stream.

The bottom width (20 ft) was estimated from the Albion survey. The side slopes were assumed to be 1:1 based on both the Albion survey and a site visit.

Table 6. TR-55 Model Assumptions and Parameters	
CN = 69	
Time of concentration details	Shallow Concentrated Flow
	L = 16,500 ft
	Slope (ft/ft) = 0.09
	Manning's n: unpaved condition
Storm Data	Type IA
	24 hr data for various return periods
No Structure Data Included	
Reach Data	Manning's n: 0.030
	Friction slope: 0.090
	Bottom width = 20 ft
	Avg. side slopes = 1:1

8. RESULTS

Overall, the peak discharge rates computed with HMS are lower than the values obtained in the NRCS/DWR TR-55 modeling effort. The NRCS/DWR TR-55 modeling run did not take into account the dam or the spillway. Both of these structures were included in the HMS model.

HMS was also run with the SCS method to see if the TR-55 data could be reproduced. Data from the HMS-SCS run is lower for the 2-, 10-, 25-, and 50-year cases, but it is higher for the 100-year case. The 2-year return period peak flow rate is exceptionally low compared to the other data sets. The low number may be the result of assuming very dry conditions for the watershed in the 2-year run. The result, 52 cfs, would indicate that the majority of the rainfall is lost to the watershed and very little becomes runoff to Upper York Creek.

The TR-55 model was run by USACE using similar assumptions from the NRCS/DWR model. The results are listed in Table 7. The USACE TR-55 results are lower than those obtained by NRCS/DWR. The percent difference has been included in the last row of the table. Differences may be attributed to slightly different assumptions and input variables between the two modeling runs.

Table 7. DWR and USACE Flow Data Comparison					
Return Period (yrs)	**2-Yr**	**10-Yr**	**25-Yr**	**50-Yr**	**100-Yr**
NRCS/DWR TR-55 Model (cfs)	380	762	1016	1285	1424
USACE TR-55 Model (cfs)	139	583	855	1066	1282
Percent Difference (%)	173%	30%	19%	20%	11%

Rainfall data used by both NRCS/DWR and USACE is listed in Table 8. The rainfall data used by NRCS/DWR as input to the TR-55 model was obtained from National Oceanic and Atmospheric Administration precipitation maps. NRCS/DWR did not take into account soil moisture conditions for different events. As mentioned in Section 6.2, the USACE modeling run used hypothetical storm conditions based on statistically derived data.

Table 8 below shows the peak discharge rates in cubic feet per second for various return periods at the dam.

Table 8. York Creek Peak Discharge Rates (cfs) Below Dam and 24 Hr Rainfall Data At Dam

Return Period (yrs)	**2-Yr**	**10-Yr**	**25-Yr**	**50-Yr**	**100-Yr**
HMS Hypothetical Storm, USACE (cfs)	116	423	707	960	1281
HMS SCS Method, USACE (cfs)	52	314	636	998	1626
Rainfall (in), SCS/DWR	4.8	6.5	7.5	8.5	9.0
Rainfall (in), USACE	3.4	5.6	6.8	7.6	8.4

The HMS data computed using a hypothetical storm event was compared to several other data sets for the surrounding area. The data are listed in Table 9. Data sets were available for the Napa River near St. Helena and for Santa Rosa Creek. The Napa River data was generated using stream gage data from 1930 to 1986. The analysis was based on the USGS annual peak flow frequency analysis following WRC Guidelines Bulletin 17-B. The Santa Rosa Creek data was calculated using a hypothetical storm event. The precipitation data used in this study is greater than the precipitation data used for the Upper York Creek analysis. This data is listed in Table 8 because Santa Rosa Creek has a similar basin as compared with Upper York Creek. Although the Santa Rosa Creek data is higher than the Napa River data and the HMS data for certain return periods, the HMS results compare favorably with these data sets.

Table 9. Flow Rate Comparison With Other Basins

Basin	**Drainage Area (mi^2)**	**2-Yr (cfs)**	**10-Yr (cfs)**	**25-Yr (cfs)**	**50-Yr (cfs)**	**100-Yr (cfs)**	**100-Yr CSM (cfs/mi^2)**
York Creek at Dam, HMS Hypothetical Storm, USACE	2.48	116	423	707	960	1281	517
Napa River Area, USACE	81.4	189	413	501	566	643	295
Santa Rosa Creek, USACE	2.2	353	707	884	1013	1138	522

8.1. Peak Discharges at the Confluence of York Creek with the Napa River

Peak discharge rates were also calculated for York Creek at its confluence with the Napa River to the east of the city of St. Helena. These values were generated to develop an understanding of the flow conditions downstream of the dam and to evaluate possible flooding potential of the creek. The values are listed in Table 9. The hypothetical storm data was used as input to the HMS model.

Table 10. York Creek Peak Discharge Rates at Confluence with Napa River (Drainage area is 5.01 mi^2)

Return Period (yrs)	Peak Discharge of York Creek at Napa River (cfs)
2	296
10	956
25	1437
50	1846
100	2306

9. CONCLUSIONS

Basin hydrology for the York Creek watershed was developed for the Upper York Creek Dam Removal Project. Modeling efforts included HMS and TR-55 runs. Results from these models were compared to areas with similar watershed characteristics and to results obtained from the SCS/DWR TR-55 modeling effort for the York Creek area. Peak discharge rates for Upper York Creek per exceedence interval are listed below. The data listed in Table 10 were calculated using a hypothetical storm as input to the HMS model. The dam has essentially no storage and offers no flood protection.

Table 11. Final Design Discharges for Existing Conditions Using the HMS Hypothetical Storm Method.

Storm Frequency (yrs)	Peak Discharge Rates at the Dam (cfs) (Drainage area is 2.48 mi^2)	Peak Discharge Rates at the Napa River (cfs) (Drainage area is 5.01 mi^2)
2	116	296
10	423	956
25	707	1437
50	960	1846
100	1281	2306

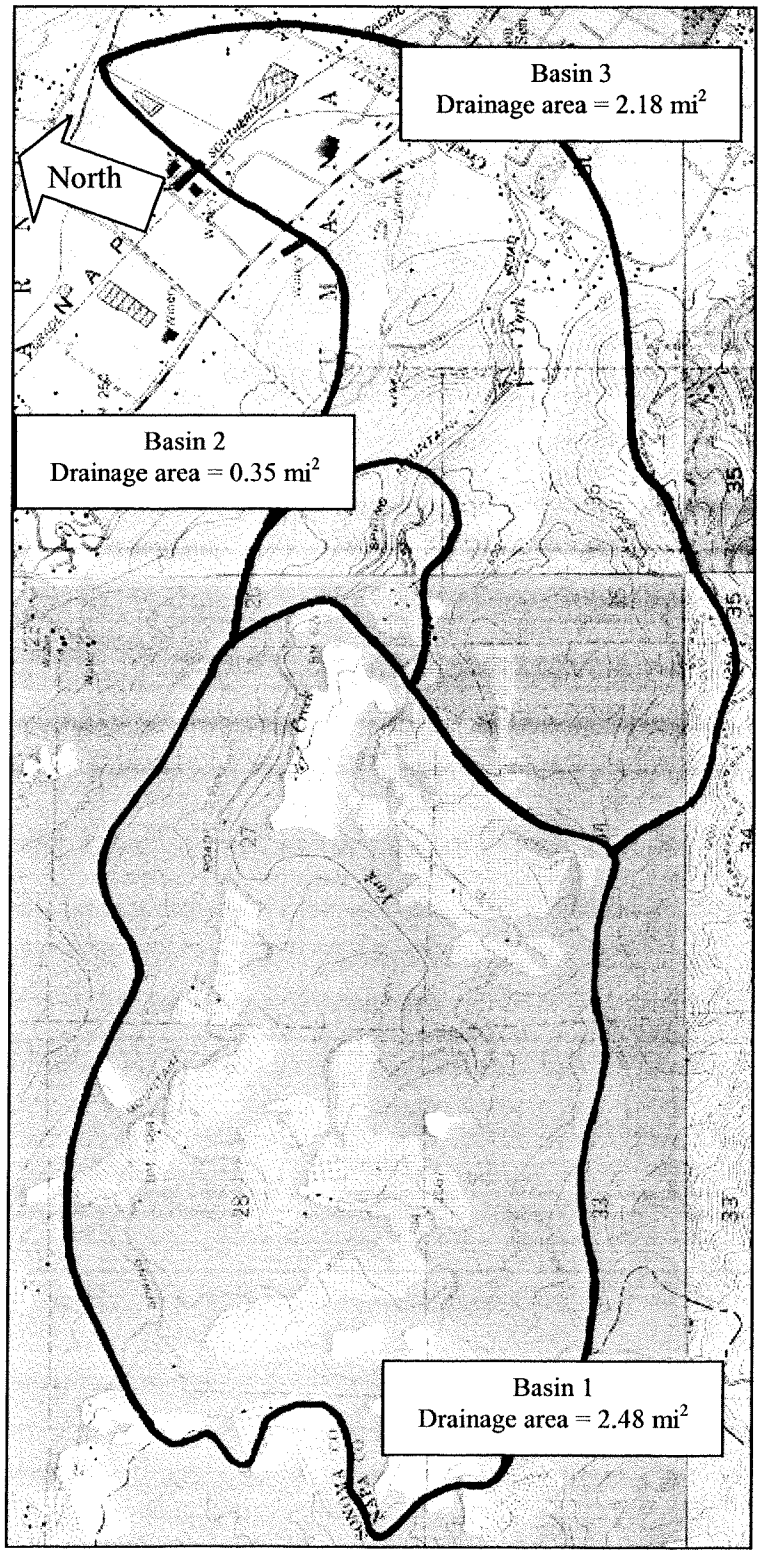
10. REFERENCES

Chow, Ven Te, David R. Maidment, and Larry W. Mays. *Applied Hydrology*. Mc-Graw Hill, 1998.

Department of Water Resources. *Initial Study/Proposed Mitigated Negative Declaration for the York Creek Dam Removal and Stream Restoration Project, Napa County, California*. July 2002.

U.S. Army Corps of Engineers. *Hydrologic Modeling System, HEC-HMS, Technical Reference Manual*. Hydrologic Engineering Center, March 2002.

APPENDIX A
FIGURES



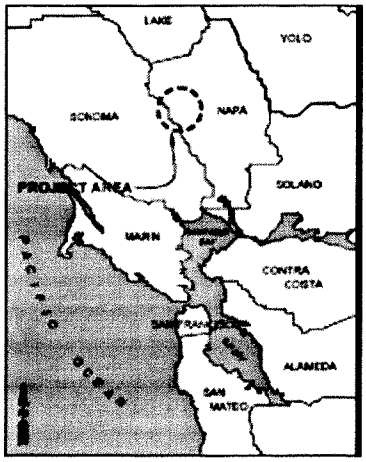


Figure 1. Project site and watershed.

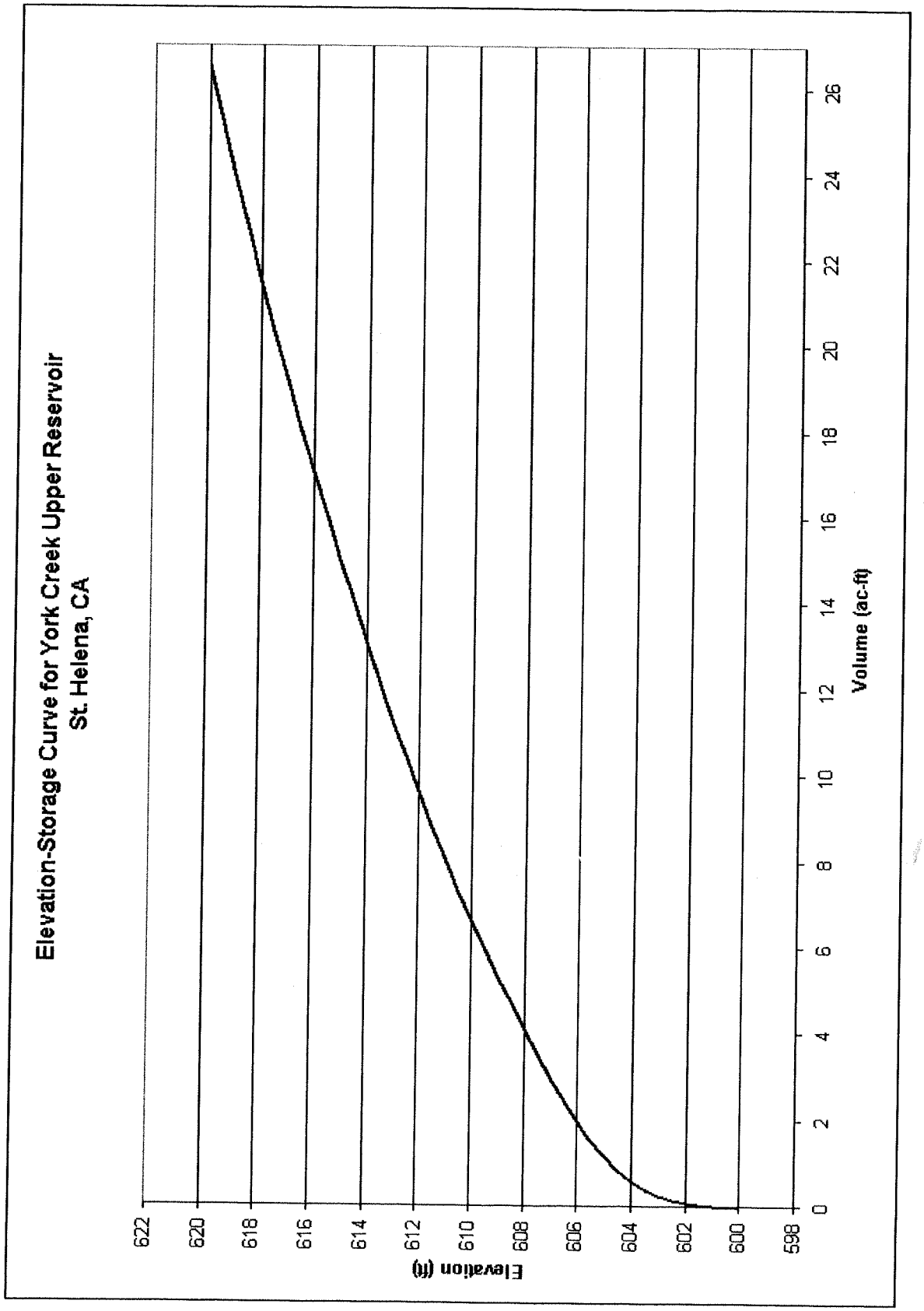


Figure 2. Elevation-storage curve.

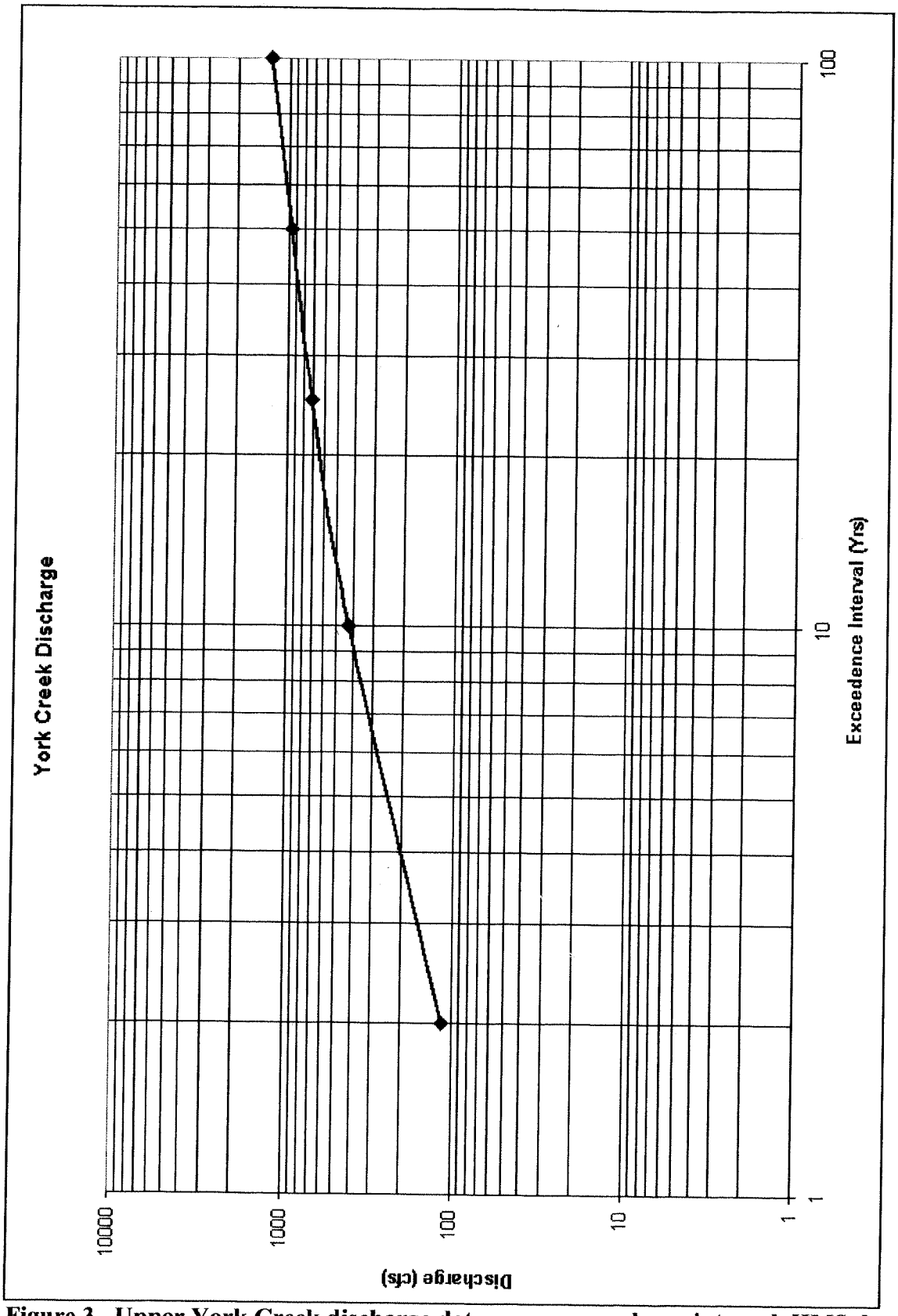


Figure 3. Upper York Creek discharge data versus exceedence interval, HMS data.

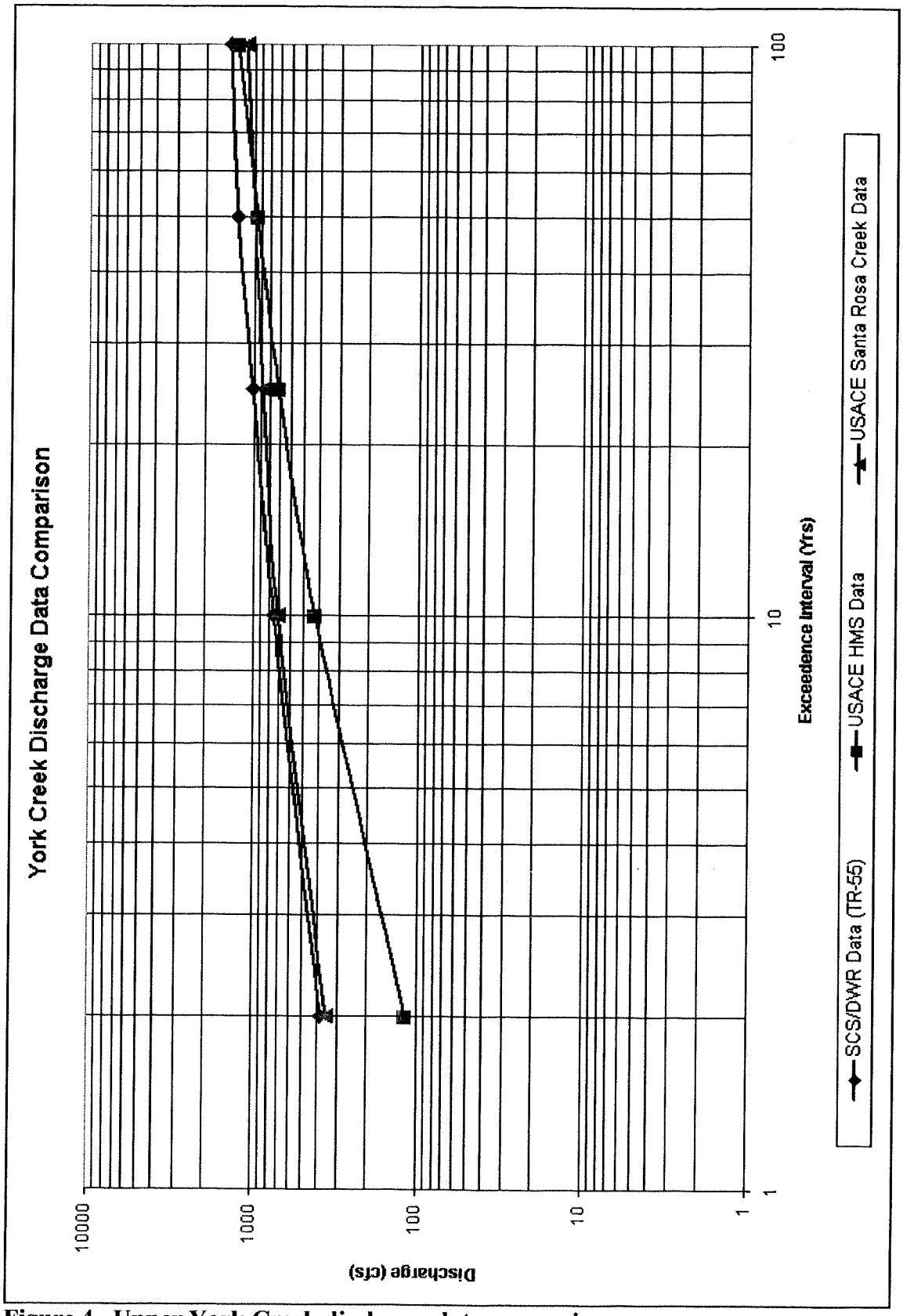


Figure 4. Upper York Creek discharge data comparison.

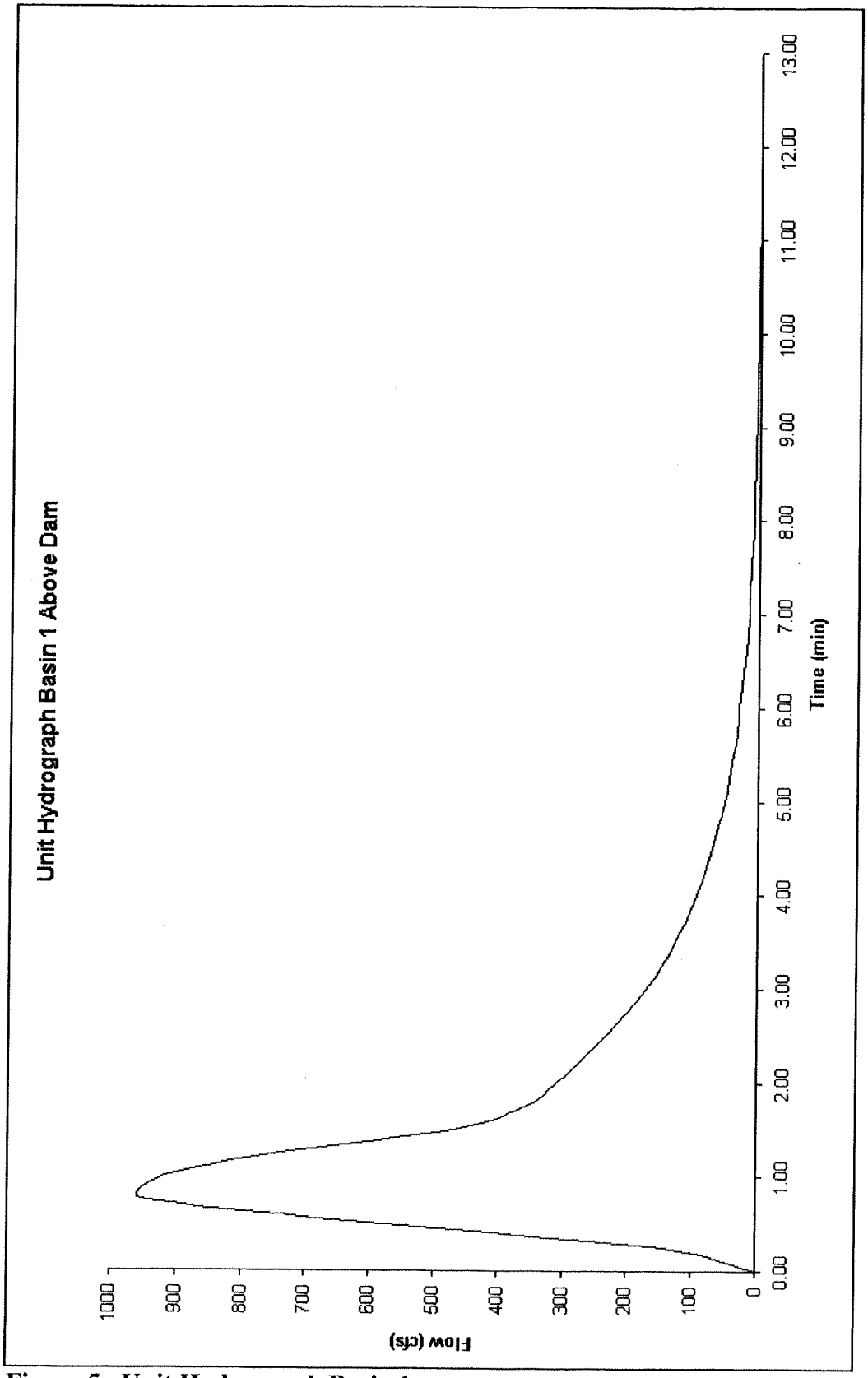


Figure 5. Unit Hydrograph Basin 1

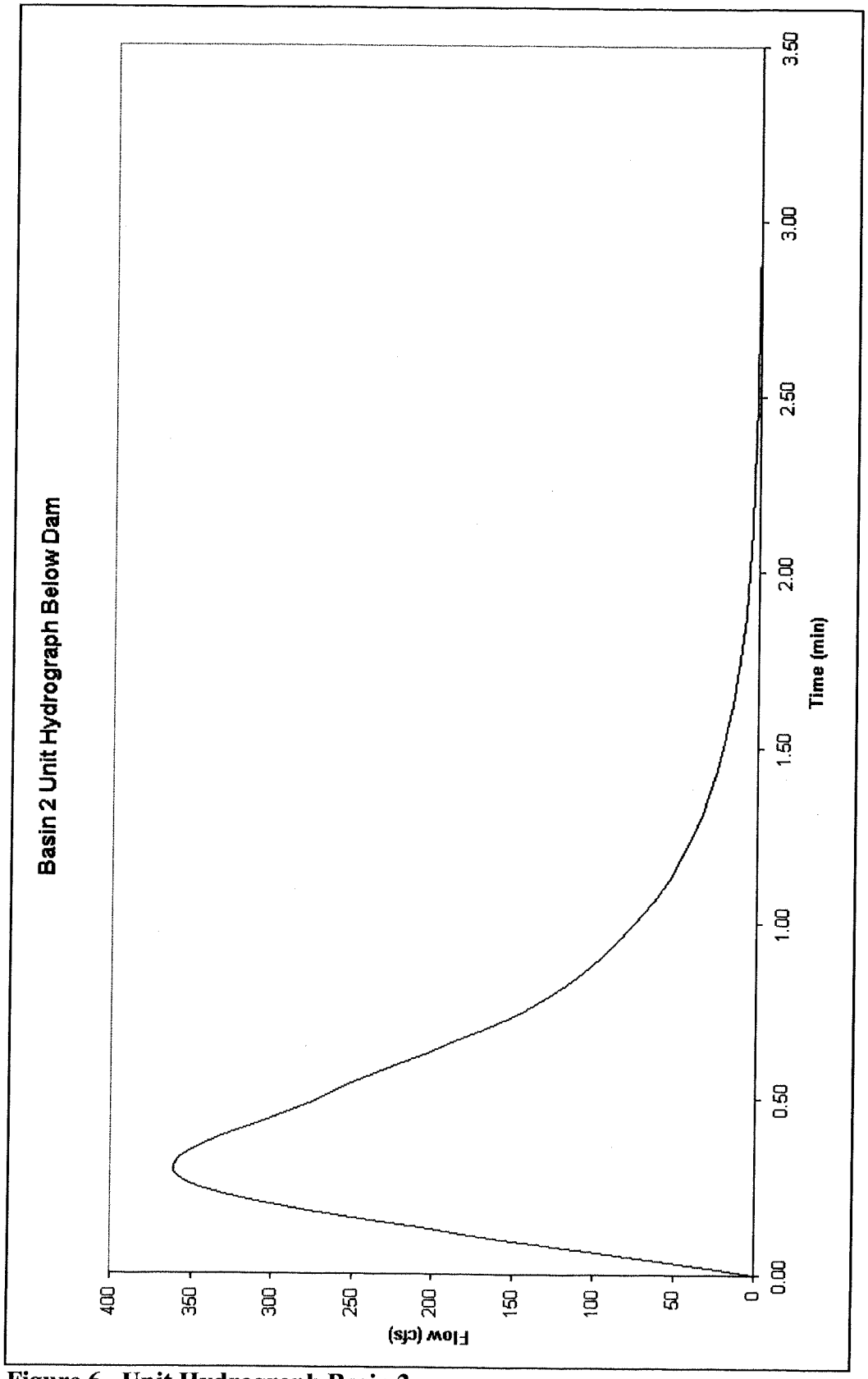


Figure 6. Unit Hydrograph Basin 2

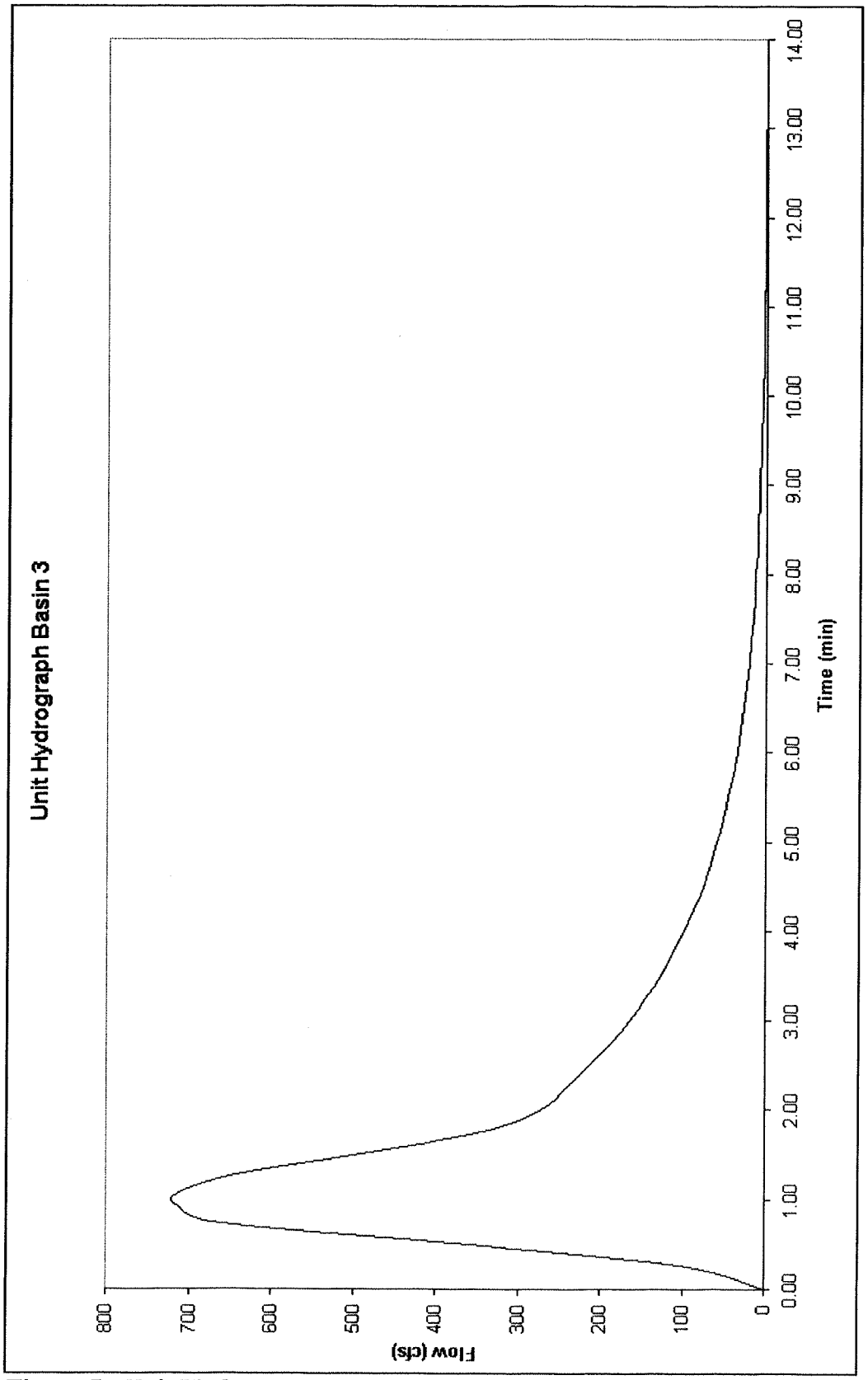


Figure 7. Unit Hydrograph Basin 3

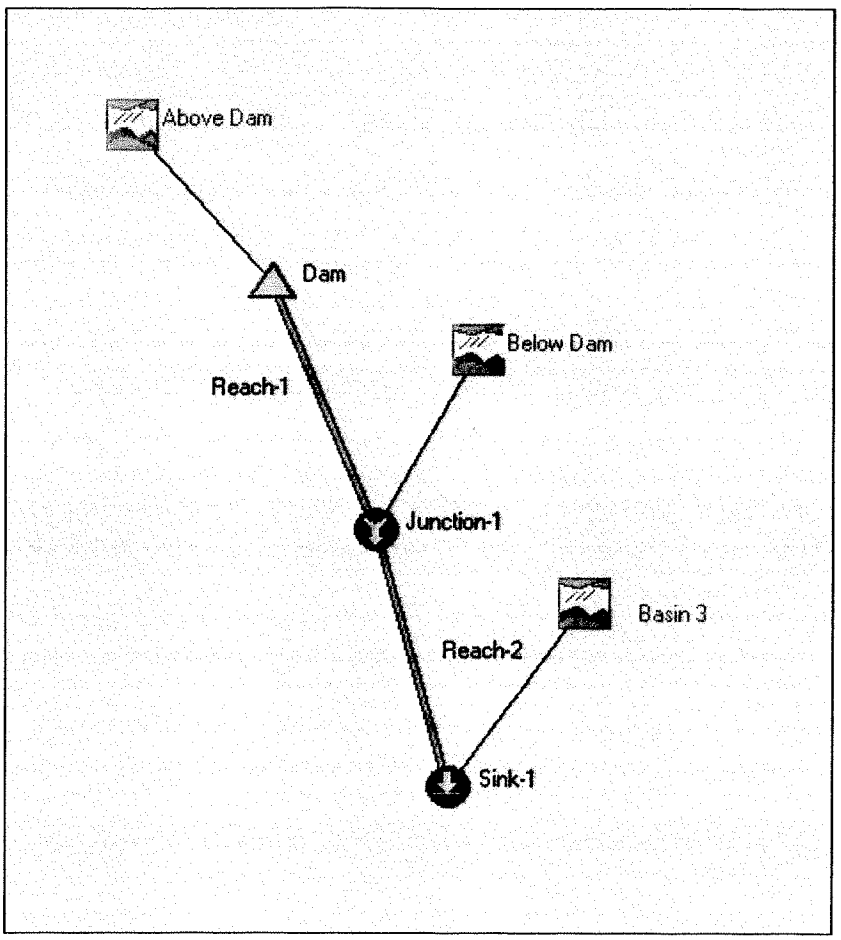


Figure 8. HMS Basin Model Setup.

Detailed Project Report

FINAL DRAFT

Upper York Creek

Ecosystem Restoration Project
Napa County, California

U.S. Army Corps of Engineers, San Francisco District, South Pacific Division
City of St. Helena, Napa County, California
December 15, 2006

York Creek Steelhead Carrying Capacity Model - Napa County RCD (March, 2006)

Habitat survey data collected in 2003 by NCRCD were compiled for the reaches above York Creek dam to the end of anadromy at a bedrock falls. These data were used to calculate summary statistics for usable habitat area for juvenile steelhead rearing. Average widths and depths were calculated and assigned to each habitat unit for the reach to arrive at total available habitat. Steelhead densities calculated from electrofishing efforts by Stillwater Sciences (2005) were assigned to each habitat category to estimate potential carrying capacity. High and low density estimates represent the highest and lowest recorded value respectively. Moderate estimates are the average of the two.

HABITAT	Above Restored Area (sq ft)	Habitat+Restored Habitat (sq feet)	Acres
TOTAL POOL AREA (sq. ft.)	11,053	13153	0.30
TOTAL FLATWATER AREA (sq. ft.)	13,016	14297	0.33
TOTAL RIFFLE AREA (sq. ft.)	34,705	36994	0.58
	58,774	**64444**	**1.21**

FISH DENSITY (# of steelhead per square foot)			
	High	Moderate*	Low
Pool	0.053	0.0375	0.022
Flatwater	0.021	0.015	0.009
Riffle	0.022	0.0165	0.011

** Calculated values*

HABITAT	length (ft)	Habitat+Restored Habitat (feet)	Acres
TOTAL STREAM LENGTH (ft.)	8,030	8855	2

ESTIMATED CARRYING CAPACITY			
	HIGH	**MODERATE**	**LOW**
POOL	697	493	289
FLATWATER	300	214	129
RIFFLE	814	610	407
TOTAL STANDING CROP	**1,811**	**1,318**	**825**
STEELHEAD PER 100 ft.	**23**	**16**	**10**

CALIFORNIA SALMONID STREAM HABITAT RESTORATION MANUAL

FOURTH EDITION

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State of California
The Resources Agency
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PART XI. **RIPARIAN HABITAT RESTORATION**

PART XII . **FISH PASSAGE DESIGN & IMPLEMENTATION**

ADAPTING TO RISING TIDES WHITE PAPER
JUNE 2012

ADDRESSING SOCIAL VULNERABILITY AND EQUITY IN CLIMATE CHANGE ADAPTATION PLANNING

Report prepared as part of the Adapting to Rising Tides project, a collaborative effort of local, state and federal agencies and organizations, and is being led by the San Francisco Bay Conservation and Development Commission. More information about this project can be found on the web at: www.adaptingtorisingtides.org

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Acknowledgements

Many thanks go to the numerous experts who provided valuable input and perspective during development and writing of this white paper. Catalina Garzon of the Pacific Institute, Jennifer Yeamans of the Metropolitan Transportation Commission and Julie Ekstrom of UC Berkeley patiently provided support with development and dissemination of the survey and review edits on the white paper. Special thanks go to Lindy Lowe and Sara Polgar for editing several drafts of the white paper. Finally, many Adapting to Rising Tides working group members helped to disseminate and provided helpful input along the way.

Cover photos credit: ABAG

San Francisco Bay Conservation and Development Commission

NOAA Coastal Services Center
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**Pacific Southwest
Research Station**

General Technical
Report PSW-GTR-144-www

Handbook of Field Methods for Monitoring Landbirds

C. John Ralph
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Peter Pyle
Thomas E. Martin
David F. DeSante

Ralph, C. John; Geupel, Geoffrey R.; Pyle, Peter; Martin, Thomas E.; DeSante, David F. 1993. **Handbook of field methods for monitoring landbirds.** Gen. Tech. Rep. PSW-GTR-144-www. Albany, CA: Pacific Southwest Research Station, Forest Service, U.S. Department of Agriculture; 41 p.

This handbook is a compilation of methods that can be used to assay population size, demographics, and status of many species of birds occurring in a wide variety of habitats. The handbook will prove useful to field biologists, managers, and scientists anywhere in the New World from the arctic through the tropics. The methods include four types of censuses for determining population size and trends, mist-netting and nest searches to determine demographic parameters, and other methods that will be useful in operating a monitoring station, including habitat and weather observations, and suggestions for training personnel and possibilities for detailed studies. Suggestions of specific methods and data forms are included.

Retrieval Terms: bird populations, census, mist-nets, monitoring, nesting birds

Acknowledgments:

The late L. Richard Mewaldt was the first among equals in setting high standards and maintenance of accurate records. His contributions are detailed by Ralph (1992). We take great pleasure in dedicating this handbook to him.

This handbook is a direct outgrowth of the landbird program started more than 25 years ago at the Point Reyes Bird Observatory to monitor landbird populations in coastal California, and much of this handbook is the result of the methods developed and adapted there. Over this period, many people contributed to the development of this landbird monitoring program.

The handbook benefitted greatly from discussions and correspondence by Bruce Bingham, Grant Ballard, Danny Bystrak, Barbara Carlson, Brenda Dale, Sam Droege, John Faaborg, Kevin J. Gutzwiller, Denise Hardesty, Kimberly Hollinger, Bill Howe, David W. Johnston, Stephanie Jones, Cherry Keller, Kathy Klimkiewicz, Rolf R. Koford, Karin Kozie, Borja Milá, Sherri Miller, Michael Morrison, Barry R. Noon, Nadav Nur, Raymond J. O'Connor, Will Peach, Carol Pearson Ralph, Martin Raphael, Dan Reinking, Christopher C. Rimmer, Sharon Ritter, John T. Rotenberry, John R. Sauer, Tom Sherry, Sue Sniado, John Tautin, Jared Verner, Dennis Vroman, George E. Wallace, Dan Welsh, and Joseph M. Wunderle, Jr.

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Pacific Southwest Research Station
Albany, California
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May 1993

Handbook of Field Methods for Monitoring Landbirds

C. John Ralph Geoffrey R. Geupel Peter Pyle Thomas E. Martin David F. DeSante

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Riparian Zone Monitoring Plan

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August 2010

Acknowledgements

Thank you to the San Francisco Bay Regional Water Quality Control Board, State Water Resources Control Board and American Recovery & Reinvestment Act for providing the financial resources to produce this document. Our gratitude goes to the many landowners and land managers who invested time and money to voluntarily implement conservation practices over the year while providing local food. Many generously shared information about the history of specific changes to local streams and riparian areas as well as practical and theoretical questions regarding the long-term outcomes from natural resource conservation. We truly appreciate the life-long stewardship and commitment to the land that is common among community residents.

Contributors

This plan was the result of contributions from numerous local professionals. Specifically, the following partnering organizations helped to produce this document:

- USDA Natural Resources Conservation Service
- The Bay Institute's Students & Teachers Restoring A Watershed
- Point Reyes Bird Observatory Conservation Science
- Prunuske Chatham, Inc.
- Jeff Creque, Certified Rangeland Manager
- Marin Agricultural Land Trust
- San Francisco Bay Regional Water Quality Control Board
- Southern Sonoma County Resource Conservation District
- California Department of Fish and Game
- Point Reyes National Seashore
- Marin Municipal Water District
- The Nature Conservancy

Suggested Citation

Lennox, M., N. Scolari, and D. Lewis. 2010. Riparian Zone Monitoring Plan. Prepared by University of California Cooperative Extension for Marin Resource Conservation District, Point Reyes Station CA. 75 p.

Project Summary

The conservation of natural resources and agricultural viability has been the focus of the Marin Resource Conservation District (RCD) for 50 years. Delivering financial and technical support with partnering organizations to landowners has evolved over the last half century in the face of increased demand for stream restoration and locally produced food. Responding to societal needs and explaining the long-term outcomes from natural resource enhancement efforts has continued to be a challenge for agriculture.

Assisting landowners to meet their needs has changed as watershed and creek management issues, such as water quality, are increasingly problematic. Solutions for controlling erosion and managing the corridor along streams have improved since the 1970's. Marin RCD and its partners have pioneered advances in riparian restoration technology and now provide concise, scientific approaches to watershed restoration based on site conditions.

This Riparian Zone Monitoring Plan (RZMP) is for conservation projects implemented in riparian areas targeted in watershed recovery efforts to control erosion and sedimentation, increase aquatic, riparian, and upland habitat and stabilize eroding stream channels. The RZMP applies to any stream from headwater creeks or gullies to large streams or small rivers. Its goal is to provide funding and permitting agencies the confidence that projects are systematically monitored while guiding Marin RCD staff and partners to efficiently collect and report monitoring results for integration with the Permit Coordination Program (MRCD 2004).

Overall, the RZMP provides a science-based guide to organize post-project monitoring based on site-specific objectives to further understand agricultural sustainability and ecosystem services. It standardizes monitoring protocols and prioritizes questions for periodic evaluation. Consistent and systematic monitoring of project outcomes will continue to improve conservation practices while maintaining landowner confidentiality. Marin RCD's watershed restoration program is built upon the hard work by community residents, landowners, ranchers, farmers, consultants, restoration practitioners, agencies, scientists, oyster growers, and other stakeholders.

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The Riparian Bird Conservation Plan

A strategy for reversing the decline of riparian associated birds in California

A project of California Partners in Flight and the Riparian Habitat Joint Venture

The Riparian Bird Conservation Plan

A strategy for reversing the decline of riparian associated birds in California

Version 2.0

2004

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Bureau of Reclamation
Point Reyes Bird Observatory
Packard Foundation
National Fish and Wildlife Foundation

Publication

Bureau of Reclamation
PRBO Conservation Science

Meeting Facilitator

Dave Ceppos, Jones & Stokes Associates, Inc.

Recommended Citation

RHJV (RIPARIAN HABITAT JOINT VENTURE). 2004. Version 2.0. The riparian bird conservation plan: a strategy for reversing the decline of riparian associated birds in California. California Partners in Flight. http://www.prbo.org/calpif/pdfs/riparian.v-2.pdf.

For copies of this plan, please contact the PRBO Conservation Science at (415) 868-0655 or write to: Riparian Conservation Plan, c/o PRBO, 4990 Shoreline Hwy., Stinson Beach, CA 94970. An electronic version of this plan is available at http://www.prbo.org/calpif/plans.html.

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The San Pablo Bay Watershed Restoration Program

DRAFT
Watershed Management Plan

Photo credit: EPA, Paul Kelly

Prepared by:

The U.S. Army Corps of Engineers
San Francisco District
May 6, 2010

In coordination with:

The Bay Institute

San Pablo Bay Watershed Management Plan
DRAFT
May 6, 2010

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DRAFT

Why Climate Change Makes Riparian Restoration More Important than Ever: Recommendations for Practice and Research

Nathaniel E. Seavy, Thomas Gardali, Gregory H. Golet, F. Thomas Griggs, Christine A. Howell, Rodd Kelsey, Stacy L. Small, Joshua H. Viers and James F. Weigand

ABSTRACT

Over the next century, climate change will dramatically alter natural resource management. Specifically, historical reference conditions may no longer serve as benchmarks for restoration, which may foster a "why bother?" attitude toward ecological restoration. We review the potential role for riparian restoration to prepare ecological systems for the threats posed by climate change. Riparian ecosystems are naturally resilient, provide linear habitat connectivity, link aquatic and terrestrial ecosystems, and create thermal refugia for wildlife: all characteristics that can contribute to ecological adaptation to climate change. Because riparian systems and the projected impacts of climate change are highly variable geographically, there is a pressing need to develop a place-based understanding of climate change threats to riparian ecosystems. Restoration practitioners should consider how they can modify practices to enhance the resilience of riparian ecosystems to climate change. Such modifications may include accelerating the restoration of private lands, participating in water management decisions, and putting the emerging field of restoration genetics into practice.

Keywords: California, climate change, genetics, hydrology, restoration, riparian

In recent decades, advances in the science and practice of restoring damaged riparian ecosystems have addressed the local and regional threats posed by habitat loss and non-native species invasions (Bernhardt et al. 2005). While ecological restoration has often emphasized a return to historical reference conditions, this target has been complicated in riparian systems, where flood control and water delivery often result in modifications of natural flows. Today, riparian restoration is further complicated by global climate change (Harris et al. 2006, Battin et al. 2007, Palmer et al. 2008). During the next century, global temperatures are projected to increase by 1.1 to 6.4°C (IPCC 2007a, 2007b). Riparian ecosystems will face increases in air and surface water temperatures, alterations in the magnitude and seasonality of precipitation and run-off, and shifts in reproductive phenology and distribution of plants and animals (Meyer et al. 1999, Barnett et al. 2005, Parmesan 2007, Palmer et al. 2008, Rosenzweig et al. 2008). In this context it is now clearer than ever that a return to historical reference conditions will no longer be the benchmark for restoration success (Choi et al. 2008, Seastedt et al. 2008).

Given the uncertainties about future conditions, climate change may cause people to ask "why bother with restoration?" This question has motivated us to reevaluate our work in the science and practice of riparian restoration. Society is becoming aware of the need for mitigation and adaptation to address the adverse impacts of anthropogenic climate change. Mitigation describes actions for reducing greenhouse gas concentrations, for example, by reducing emissions or developing sinks that remove these gasses from the atmosphere (IPCC 2007a). Adaptation refers to actions designed to reduce the vulnerability of natural and societal systems to the effects of climate change (IPCC 2007a). Even if mitigation efforts were to stop the increase in all greenhouse gas emissions, adaptation would remain important because greenhouse gases already in the atmosphere today will continue to cause the climate to change for decades (IPCC 2007b, Solomon et al. 2009).

Climate change adaptation strategies often propose activities that can enhance ecological resilience (Millar et al. 2007, Heller and Zavaleta 2008). Ecological resilience encompasses 1) the amount of disturbance a system can withstand before changing state; 2) the rate at which a system recovers after disturbance; and 3) the way in

Ecological Restoration Vol. 27, No. 3, 2009
ISSN 1522-4740 E-ISSN 1543-4079
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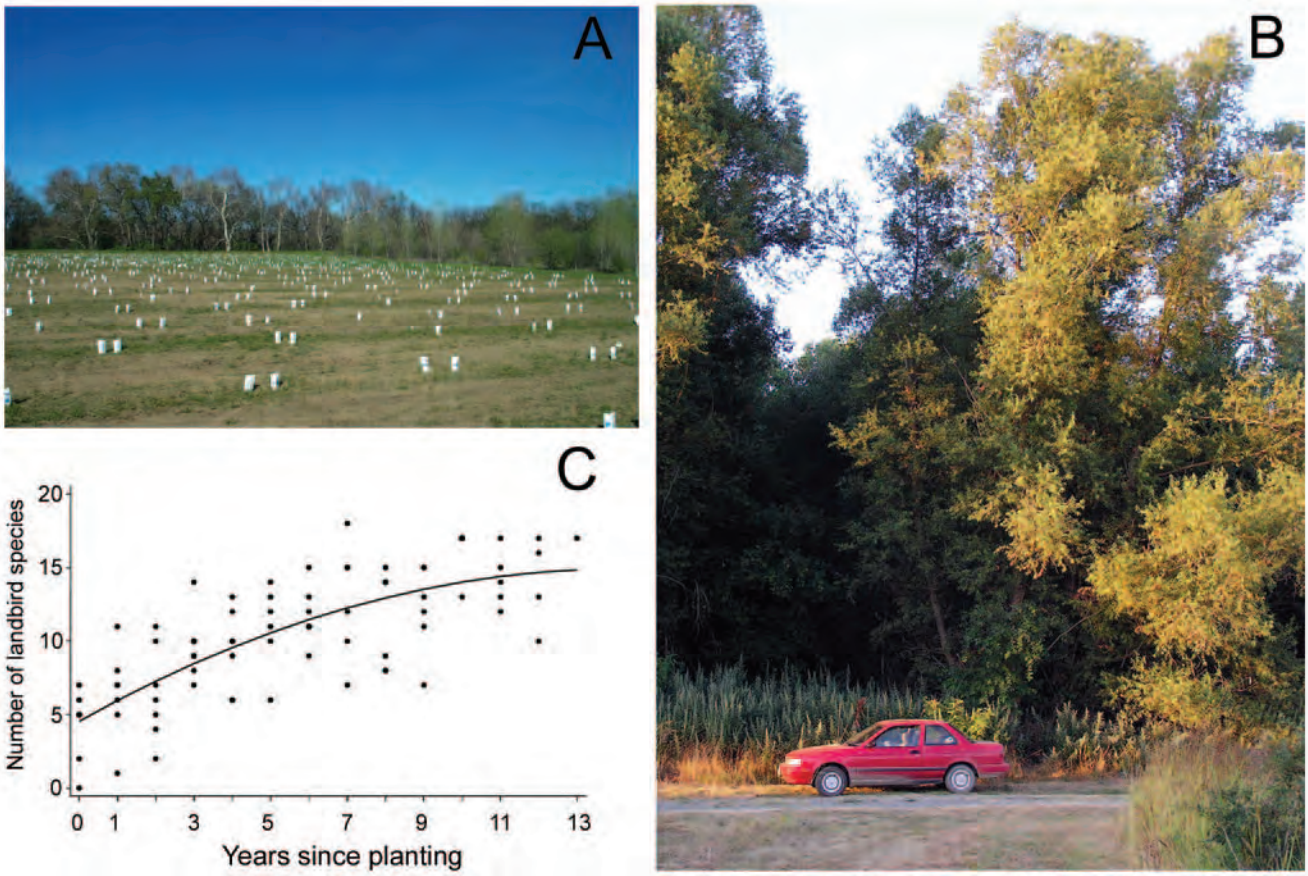

Figure 1. On the Sacramento River in California's Central Valley, horticulture-based restoration can transform open fields (A) to well-established riparian forest (B) in as little as 13 years. Over this same time, riparian wildlife communities, as measured by such metrics as bird species richness (C) can exhibit dramatic recoveries. Examples like this illustrate the inherent resilience of riparian ecosystems. Images reprinted with permission from Gardali et al. 2006 and Golet et al. 2008

which a system responds to gradual changes (Gunderson 2000, Scheffer et al. 2001). Specific recommendations for enhancing ecological resilience, however, are lacking (Heller and Zavaleta 2008).

Based on general ecological characteristics of riparian systems and our collective experience restoring these ecosystems in California, we suggest that healthy riparian ecosystems promote ecological resilience both within and beyond riparian zones. Here we address 1) how and why riparian restoration prepares ecosystems for climate change; 2) how riparian restoration can be enhanced to accommodate climate change; and 3) research needed to ensure that riparian restoration is robust to climate change.

How and Why Riparian Restoration Prepares Ecosystems for Climate Change

For forested ecosystems, Millar and others (2007) provide examples of management practices to enhance ecosystem resilience to climate change. These practices include enhancing habitat connectivity, promoting redundancy and buffers, reducing landscape synchrony (by maintaining a mix of successional stages), realigning disrupted conditions, expecting surprises, and identifying and protecting refugia (Millar et al. 2007). Similar recommendations have been made for a wide variety of habitats (Hansen et al. 2003). We use this general framework to review five specific reasons that riparian restoration can enhance ecosystem resilience to climate change.

Natural Resilience of Riparian Systems

Climate change is projected to lead to increased frequency and severity of extreme weather events, which will likely result in more frequent and severe floods as well as more intense droughts (Easterling et al. 2000). The rate at which ecological systems recover from disturbance will be an important consideration when designing restoration activities. Many riparian plants are adapted to hydrologic and geomorphic disturbances and tolerate both seasonal and annual variation in environmental conditions (Naiman and Decamps 1997). Thus, compared to plants in adjacent upland habitats, riparian species may be more resilient to the increased flooding or drought projected for many regions (Milly et al. 2002, Seager et al. 2007). Restoration programs that reestablish

appropriate hydrological processes, actively intervene with horticultural techniques to propagate and establish native vegetation where necessary, and manage for genetic diversity to facilitate evolutionary processes can build upon the natural resilience of riparian systems.

The natural resilience of riparian systems is exhibited in the response of riparian wildlife communities to habitat restoration along the Sacramento River in central California (Figure 1). Since European settlement, 95 percent of 324,000 ha of riparian habitat along this river was lost to logging, agriculture, urban development, and flood-control and power-generation projects (Katibah 1984). In 1988, the Sacramento River Project was launched to coordinate restoration efforts along a 161 km reach of the Sacramento River between the cities of Red Bluff and Colusa (Golet et al. 2008). A major component of this project was to restore sites previously in agriculture, mostly walnut (*Juglans regia*) and almond (*Prunus dulcis*) orchards, by planting local ecotypes of indigenous tree, shrub, and understory species (Alpert et al. 1999). In just ten years, these efforts restored a broad suite of faunal species including both special-status species and the larger native riparian animal community (Gardali et al. 2006, Golet et al. 2008). Consistent with the very high growth rates of floodplain trees, species associated with mature forest, including cavity-nesting birds and crevice-roosting bats, reoccupied restoration sites within a decade (Gardali et al. 2006, Golet et al. 2008).

Of course, there are limits to resilience. Severe human-induced disruptions to these regimes can interrupt important plant and animal population processes. Furthermore, disturbances that are too far out of any given system's natural range of variability (Richter et al. 1997) may lead to the proliferation of disturbance-adapted non-native invasive species (Zedler and Kercher 2004).

Enhancing Connectivity

Climate change, whether natural or anthropogenic, causes distributional shifts for many organisms. Prioritizing connectivity in landscape planning and reserve design is the most common recommendation for protecting biodiversity from climate change (Heller and Zavaleta 2008). Connectivity is also critical for preserving the ecological processes for evolutionary adaptations to climate change (Cowling and Pressey 2001).

Riverine habitats function as ecological corridors for a wide array of plants and animals (Naiman et al. 1993, Machtans et al. 1996, Hilty and Merenlender 2004). Rivers and riparian vegetation connect high-elevation montane areas to sea-level estuaries and oceans. Rivers flow across elevational gradients, linking ecological zones with different climates. Furthermore, tributaries within watersheds provide spatial redundancy that maintains metapopulation and metacommunity dynamics (Collinge et al. 2001). In addition, flowing water moves organic material and energy (Ahearn et al. 2006, Kondolf et al. 2006). Restoring riparian habitats and hydrological function recreates or increases connectivity between habitats and across elevational zones, thus providing avenues for species movements in response to climate change.

Promoting Linkages between Aquatic and Terrestrial Systems

Riparian zones link riverine and terrestrial systems and make each more ecologically diverse and productive (Naiman and Decamps 1997). Aquatically derived nutrients support luxuriant vegetation and diverse wildlife communities in adjacent upland areas (Merz and Moyle 2006, Uesugi and Murakami 2007). Emerging aquatic insects are prey for birds and bats foraging and breeding in riparian areas (Knopf et al. 1988, Grindal et al. 1999). Equally important are the resources and services that riparian areas convey from terrestrial to aquatic systems. The terrestrial component of riparian vegetation protects water quality by trapping sediment and filtering pollutants through physical and biological processes (National Research Council 2002), and furnishes aquatic food webs with detritus for aquatic invertebrates and terrestrial insect prey for fish (Wipfli 1997). Restoring riparian habitat will strengthen linkages between aquatic and terrestrial systems, making both more resilient and resistant to the stresses imposed by climate change.

In California, the importance of linkages between aquatic and terrestrial systems is exemplified by the Yolo Bypass, an engineered floodplain on the Sacramento River. When the Yolo Bypass floods, 24,000 ha of agricultural land, wetlands, and riparian and upland vegetation are covered with shallow water. The flooding provides important benefits to aquatic, wetland, and terrestrial taxa, including fish and birds (Sommer et al. 2001).

Expanding Thermal Refugia

Climate change is projected to result in higher air temperatures and, in turn, higher surface water temperatures (Battin et al. 2007, Nelson and Palmer 2007). Because riparian areas have higher water content than surrounding upland areas, they absorb heat and buffer organisms against extreme temperatures (Naiman et al. 2000). During previous periods of climate change, riparian areas served as refugia because they provided microclimates that protected plant biodiversity (Bakker 1984, Meave and Kellman 1994). Riparian vegetation can maintain cooler water temperatures by shading water from sunlight (Sridhar et al. 2004, Cassie 2006) and the infusion of cold groundwater into warmer surface waters creates and maintains pockets of cool water (Chu et al. 2008). Thus, riparian areas provide thermal refugia for animals with thermoregulatory limitations. For example, salmon are able to successfully migrate through high temperature river reaches, but

only when reaches contain pockets of cooler water (McCullough et al. 2001). Restoring vegetation and protecting groundwater resources will enhance thermal refugia that will be increasingly important as air temperature rises.

Hydrological Benefits

The projected effects of climate change on hydrologic regimes include increased frequency of extreme flooding events and altered seasonal patterns of precipitation and run-off (Milly et al. 2002, Barnett et al. 2005, Palmer et al. 2008). For example, in California's Central Valley, summer stream flows are projected to decline as annual snowpacks diminish and melt earlier in the spring (Hayhoe et al. 2004, Vicuna and Dracup 2007). Riparian vegetation can promote water infiltration (Brauman et al. 2007) and reduce losses to the ocean as more precipitation falls as rain. However, because riparian vegetation also removes water through evapotranspiration, the net effect of riparian vegetation on water flow is complex. More research on this topic is needed, especially comparing the effects of riparian vegetation on water flows to that of alternative land uses, such as orchards and row crops (Tabacchi et al. 2000).

Restoring riparian ecosystems may also reduce the impacts of extreme flood events. Levees, especially those nearest the river channel, may increase flood stage and flow velocity during floods (Gergel et al. 2002). Riparian restoration to reconnect the river channel with its floodplain by moving back or breaching levees can benefit ecosystem function and nonstructural flood control for urban or agricultural areas (Poff 2002, Golet et al. 2006). The engineered floodplains of the Yolo Bypass on the Sacramento River in California show how restoring ecologically important riparian processes can also provide flood protection for human populations (Sommer et al. 2001). By recharging groundwater and reducing flood damage, riparian restoration will strengthen ecosystem resistance against extreme floods and altered surface flows anticipated from climate change.

Restoration Strategies and Practices That Accommodate Climate Change

The challenges facing restoration practitioners are not trivial. To meet these challenges, restoration practitioners will need to remain flexible and creative. Novel conditions created by climate change will require that restoration proceeds within the framework of adaptive management, in which specific hypotheses are tested and monitoring is used to verify that desired outcomes are achieved (O'Donnell and Galat 2008). Here, we discuss how specific aspects of on-the-ground restoration activities might be modified to accommodate climate change and to build resilience.

Horticultural Restoration Strategies

Some horticultural restoration techniques can enhance riparian ecosystem resilience. Techniques under investigation include using ecological genetics to prepare for unexpected conditions (e.g., by deliberately increasing genetic variability) and also planting early seral colonizers adapted to flooding together with late seral species that may be less tolerant of flooding but grow better on drier sites.

Currently, plant materials for restoration are often collected locally, under the assumption that genetically controlled local adaptations are advantageous. When the climate is changing rapidly, planting only local genetic material may not be the most appropriate strategy (Rice and Emery 2003, Bower and Aitken 2008). Collecting seed from within a watershed but across a range of elevations may better facilitate evolutionary adaptation to climate change. Modifying horticultural practices to account for uncertainty is one approach to ensure that riparian restoration is robust to climate change. Planting species that are associated with both ends of the hydrologic spectrum may provide some insurance against unexpected future conditions. Incorporating strategies that address uncertainty into horticultural restoration has the potential to both increase the odds of short-term restoration success and provide long-term maintenance of critical evolutionary processes.

Emphasizing Restoration of Private Lands

Restoration of private lands will contribute to the connectivity, size, and quality of riparian areas at spatial scales appropriate to the challenges of climate change. Incentive programs for funding, technical assistance, and infrastructure can help private landowners to modify land-use practices and restore native vegetation for conservation (Norton 2000, Langpap 2006). In some cases, state and federal regulations for endangered species may restrict private-lands restoration without innovative incentives. Safe Harbor Agreements can allow landowners to restore habitat for endangered species without legal responsibility for impacts during restoration or to maintain the restored habitat indefinitely (Wilcove and Lee 2004). In California's Central Valley, the U.S. Fish and Wildlife Service and Audubon California have recently established a Safe Harbor Agreement for riparian and wetland restoration projects on private lands in Yolo County. This agreement allows incidental take of federally listed valley elderberry longhorn beetles (*Desmocerus californicus dimorphus*) and giant garter snakes (*Thamnophis gigas*) associated with restoration projects that enhance habitat for these species.

Such programs play an essential role in the restoration of riparian ecosystems. Future challenges include adequate funding, economic incentives for agricultural landowners to maintain their land as open space, creating holistic design criteria, and monitoring to ensure that private

lands restoration enhances ecosystem function and wildlife habitat quality.

Promoting Water and Watershed Management Policies

While planting riparian vegetation on public and private lands provides many benefits, riparian ecosystems will not fully function with insufficient water. Because climate change is projected to affect water resources for many urban and agricultural uses (Tanaka et al. 2006, Alcamo et al. 2007, Milly et al. 2008), the social and political pressures to modify riparian systems for water storage, transport, and extraction may increase. The ecological stresses of climate change on dammed rivers are projected to be greater than on undammed rivers (Palmer et al. 2008). If societies choose to respond to climate change by building taller levees, deeper wells, and larger dams, riparian ecosystems will be put at greater risk. Restoration practitioners and ecologists must engage with decisionmakers about water management. This engagement will need to include providing information on how changes in water use will influence the outcome of riparian restoration efforts.

Areas of extreme aridity will pose particular challenges, such as in California's Mojave Desert, where groundwater, forced to the surface by tectonic faulting zones, sustains arid riparian plant and animal communities. Urban water use may draw down groundwater below the roots of desert riparian vegetation. Groundwater resources for desert riparian ecosystems are already at risk, particularly at the California–Nevada border, where burgeoning human populations are draining groundwater resources (Bunn et al. 2007). To protect investments in riparian ecosystems, restoration ecologists and water users will need to collaborate to develop policies that ensure that these systems have adequate river flows and groundwater (Boulton and Hancock 2006).

Utility and benefits of water policy complementing riparian restoration are exemplified by work on the Cosumnes River in California. The ecological integrity of this system has been compromised by levees that disrupted hydrological connectivity and by groundwater overdraft resulting in reduced flows (Fleckenstein et al. 2004). This restoration project used engineered levee breaches to restore hydrological connectivity between the floodplain and river channel, which in turn increased aquatic primary production (Ahearn et al. 2006), juvenile salmon recruitment (Jeffres et al. 2008), and geomorphic heterogeneity necessary for riparian establishment and succession (Florsheim and Mount 2002, Trowbridge 2007). However, in order to restore hydrologic connectivity, reduction of upstream groundwater pumping and surface water augmentation is also necessary (Fleckenstein et al. 2004). In addition to providing ecologically important flows to the Cosumnes, this water management approach is also projected to recharge the regional aquifer tapped by two growing urban areas (Fleckenstein et al. 2004).

Riparian Research and Management Priorities for Climate Change

Research on riparian and hydrologic function spans multiple spatial scales, from the global impacts of climate change on hydrologic patterns to the evolutionary responses of plants and animals to changing climate conditions. We pose several specific research questions for restoring riparian ecosystems in a time of climate change.

How can historic hydrology inform future projections? Recent advances in computing technology allow for new, more robust analyses of the spatial and temporal dimensions of climate change, hydrology, and riparian ecology (Vicuna and Dracup 2007). These approaches use historic information to understand natural variability and the adequacy of climate models for projecting future hydrological conditions. From a nearly 100-year record of daily average flow from the Cosumnes River in California, for example, Booth and colleagues (2006) identified eight types of water-years based on the frequency, magnitude, and duration of hydrological events. This information provides useful historical context for future hydrological conditions.

On a broader scale, climate modeling can provide detailed information about the hydrologic consequences of climate change. Synthetic rainfall-runoff models can aid in understanding how changes in air temperatures influence evapotranspiration potential and surface water runoff. By combining spatial and temporal information, emerging methods will inform practitioners about potential future conditions to improve the planning, design, and implementation of restoration.

Effects of climate change on riparian systems will vary dramatically among river systems (Palmer et al. 2008); thus ecologists, climate modelers, and restoration practitioners need to anticipate consequences of climate change within the context of local ecosystems. This process will be similar to restoring ecosystems in areas where people have disrupted natural fire regimes—a one-size-fits-all solution is impractical. Instead, practitioners will need to draw upon local knowledge of ecosystems and guiding ecological principles to develop appropriate restoration strategies and prescriptions (Fulé 2008).

How can we enhance recruitment of wildlife populations into restored areas? To better respond to rapid environmental shifts accompanying climate change, research on methods to speed recruitment of wildlife into new habitat is needed. Two overarching questions arise: 1) what factors most strongly influence the distribution of a species? and 2) what characteristics promote viable populations in novel habitats? We will need research at the local, reach-level scale, as well as at watershed and multiwatershed

scales. At the reach level, experimental designs could be developed to determine planting prescriptions or vegetation management that produces desired vegetation composition and structure most rapidly. At the largest scales, research related to effective connectivity (both within the floodplain corridor and extending to upland habitats), landscape matrix, and land use patterns is needed. These small-to-large-scale studies can address the dynamic and unpredictable nature of riparian systems caused by hydrologic regimes and resulting in a dynamic network of seral stages or "mobile habitat mosaics" (Hughes et al. 2005). Research on behavioral mechanisms driving different species' dispersal choices may have an increasingly important role to play in the restoration of wildlife habitat. For example, the use of conspecific attraction—the tendency of individuals of a species to settle near one another—could facilitate the colonization of suitable habitat (reviewed in Ahlering and Faaborg 2006). Similarly, a better understanding of the dispersal and metapopulation dynamics of target species is essential for promoting restoration that improves landscape connectivity for wildlife (Bélisle and St. Clair 2001, Collinge et al. 2001).

How will the phenology of riparian plants and animals respond to climate change? Another ecological challenge of climate change is the loss of synchrony between reproductive phenology and resource availability or natural disturbances (Both and Visser 2001, Inouye 2008). Understanding the physiological tolerances and phenological responses of riparian plants and animals to changes in climate and hydrology will become increasingly important to restoration ecologists. For example, in California's Central Valley, Oregon ash (*Fraxinus latifolia*) typically breaks bud in mid-March in the northern Sacramento Valley, while on the lower San Joaquin River its buds break on the first of May. This six- to eight-week difference in phenology holds true for seedling germination from both areas as well. Phenological variation may be associated with genetic variation. In European riparian systems, black poplars (*Populus nigra*) show significant genetic variation among populations, even within catchments (Smulders et al. 2008). Understanding how genetic and environmental differences between the watersheds contribute to phenological variation will help in developing new strategies to restore riparian vegetation in a manner that is robust to climate change (Hufford and Mazer 2003).

Conclusion

Responding to climate change at the appropriate temporal and spatial scales will require that government agencies, private land owners, and nongovernmental organizations work together to improve water policy, land management, urban development, and many other diverse matters. Governments will be formative in climate change adaptation and the organization of management to transition from adaptation to recovery. However, the speed with which national governments respond to climate change may not match the need for ecological adaptation to climate change. For example, as recently as 2007, the U.S. Department of the Interior was not yet providing guidance to resource managers about how to respond to actual or forecasted climate changes (U.S. General Accountability Office 2007). In the absence of national leadership, smaller-scale governments and nongovernmental organizations may have a large role in guiding management that prepares riparian ecosystems for climate change.

The effects of climate change are already upon us, making it more important than ever for restoration practitioners and researchers to share information effectively and with diverse audiences. Information networks, such as the Global Restoration Network (www.globalrestorationnetwork.org) and the National River Restoration Science Synthesis (nrrss.nbii.gov), will be important to ensure that the most recent findings and best management practices are shared among those monitoring, managing, and restoring riparian ecosystems. The challenge includes efficiently converting information into knowledge (Roux et al. 2006). Sharing information about restoration and climate change with policymakers and the public is necessary. In an era of rapid environmental change, we need to inform people about the likely threats that climate change poses to ecosystems and society. It is also important to empower people with the knowledge that ecological restoration has the potential to reduce the severity of these threats.

Basic challenges confronting restoration practitioners today will remain important in the future. Invasions of non-native species, facilitated by climate change and movements by people, will continue to occur. Altered hydrological regimes may increase the risk that restoration strategies that worked in the past will fail in the future.

When medical resources are limited, doctors use triage to prioritize the treatment of patients based on the urgency for care and the likelihood that treatment will be successful. In a management context, Millar and colleagues (2007) suggested that such an approach to preparing ecosystems for climate change could be used "to sort management situations into categories according to urgency, sensitivity, and capacity of available resources to achieve desired goals." Given that many riparian systems are highly degraded from a long history of anthropogenic activities (Tockner and Stanford 2002, Zedler and Kercher 2005), we are now faced with a decision about whether to continue investing resources to treat these ecosystems. Functional riparian systems have tremendous potential to reduce the adverse effects of climate change by enhancing ecosystem resilience. To benefit from this capacity, we urgently

need riparian restoration and the science that guides it.

Acknowledgments

This manuscript was funded in part by a CALFED Science Fellowship (U-04-SC-005) to N.E. Seavy and a grant from the S.D. Bechtel Jr. Foundation. Contributions to the manuscript from PRBO Conservation Science were written at the Palomarin Field Station, which received support from NSF (DBI-0533918). This is PRBO Conservation Science contribution number 1666. Comments from three anonymous reviewers improved this manuscript.

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Sonoma County General Plan 2020

OPEN SPACE AND RESOURCE CONSERVATION ELEMENT

Sonoma County Permit and Resource Management Department
2550 Ventura Avenue
Santa Rosa, CA 95403

Adopted by Resolution No. 08-0808
of the Sonoma County Board of Supervisors
September 23, 2008

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Solano Conservation and Restoration Manual

A rural landowner's guide to the conservation and enhancement of natural resources

CAFF
Community Alliance with Family Farmers

A project of
The Community Alliance with Family Farmers The Solano Resource Conservation District
2005

Solano RCD

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Standard Operating Procedure (SOP) 4.2.1.4

Stream Photo Documentation Procedure

(CARCD 2001, Written by TAC Visual Assessments work group)

Introduction:

Photographs provide a qualitative, and potentially semi-quantitative, record of conditions in a watershed or on a water body. Photographs can be used to document general conditions on a reach of a stream during a stream walk, pollution events or other impacts, assess resource conditions over time, or can be used to document temporal progress for restoration efforts or other projects designed to benefit water quality. Photographic technology is available to anyone and it does not require a large degree of training or expensive equipment. Photos can be used in reports, presentations, or uploaded onto a computer website or GIS program. This approach is useful in providing a visual portrait of water resources to those who may never have the opportunity to actually visit a monitoring site.

Equipment:

Use the same camera to the extent possible for each photo throughout the duration of the project. Either 35 mm color or digital color cameras are recommended, accompanied by a telephoto lens. If you must change cameras during the program, replace the original camera with a similar one comparable in terms of media (digital vs. 35 mm) and other characteristics. A complete equipment list is suggested as follows:

Required:

- Camera and backup camera
- Folder with copies of previous photos (do not carry original photos in the field)
- Topographic and/or road map
- Aerial photos if available
- Compass
- Timepiece
- Extra film or digital disk capacity (whichever is applicable)
- Extra batteries for camera (if applicable)
- Photo-log data sheets or, alternatively, a bound notebook dedicated to the project
- Yellow photo sign form and black marker, or, alternatively, a small black board and chalk

Optional:

- GPS unit
- Stadia rod (for scale on landscape shots)
- Ruler (for scale on close up views of streams and vegetation)
- Steel fence posts for dedicating fixed photo points in the absence of available fixed landmarks

How to Access Aerial Photographs:

Aerial Photos can be obtained from the following federal agencies:

USGS Earth Science Information Center
507 National Center
12201 Sunrise Valley Drive
Reston, VA 22092
800-USA-MAPS

USDA Consolidated Farm Service Agencies
Aerial Photography Field Office
222 West 2300 South
P.O. Box 30010
Salt Lake City, UT 84103-0010
801-524-5856

Cartographic and Architectural Branch
National Archives and Records Administration
8601 Adelphi Road
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Roles and Duties of Team:

The team should be comprised of a minimum of two people, and preferably three people for restoration or other water quality improvement projects, as follows:

1. Primary Photographer
2. Subject, target for centering the photo and providing scale
3. Person responsible for determining geographic position and holding the photo sign forms or blackboard.

One of these people is also responsible for taking field notes to describe and record photos and photo points.

Safety Concerns:

Persons involved in photo monitoring should **ALWAYS** put safety first. For safety reasons, always have at least two 2 volunteers for the survey. Make sure that the area(s) you are surveying either are accessible to the public or that you have obtained permission from the landowner prior to the survey.

Some safety concerns that may be encountered during the survey include, but are not limited to:

- Inclement weather
- Flood conditions, fast flowing water, or very cold water
- Poisonous plants (e.g.: poison oak)
- Dangerous insects and animals (e.g.: bees, rattlesnakes, range animals such as cattle, etc.)
- Harmful or hazardous trash (e.g.: broken glass, hypodermic needles, human feces)

We recommend that the volunteer coordinator or leader discuss the potential hazards with all volunteers prior to any fieldwork.

General Instructions:

From the inception of any photo documentation project until it is completed, always take each photo from the same position (photo point), and at the same bearing and vertical angle at that photo point. Photo point positions should be thoroughly documented, including photographs taken of the photo point. Refer to copies of previous photos when arriving at the photo point. Try to maintain a level (horizontal) camera view unless the terrain is sloped. (If the photo can not be horizontal due to the slope, then record the angle for that photo.) When photo points are first being selected, consider the type of project (meadow or stream restoration, vegetation management for fire control, ambient or event monitoring as part of a stream walk, etc.) and refer to the guidance listed on *Suggestions for Photo Points by Type of Project*.

When taking photographs, try to include landscape features that are unlikely to change over several years (buildings, other structures, and landscape features such as peaks, rock outcrops, large trees, etc.) so that repeat photos will be easy to position. Lighting is, of course, a key ingredient so give consideration to the angle of light, cloud cover, background, shadows, and contrasts. Close view photographs taken from the north (i.e., facing south) will minimize shadows. Medium and long view photos are best shot with the sun at the photographer's back. Some artistic expression is encouraged as some photos may be used on websites and in slide shows (early morning and late evening shots may be useful for this purpose). Seasonal changes can be used to advantage as foliage, stream flow, cloud cover, and site access fluctuate. It is often important to include a ruler, stadia rod, person, farm animal, or automobile in photos to convey the scale of the image. Of particular concern is the angle from which the photo is taken. Oftentimes an overhead or elevated shot from a bridge, cliff, peak, tree, etc. will be instrumental in conveying the full dimensions of the project. Of most importance overall, however, is being aware of the goal(s) of the project and capturing images that clearly demonstrate progress towards achieving those goal(s). Again, reference to *Suggestions for Photo Points by Type of Project* may be helpful.

If possible, try to include a black board or yellow photo sign in the view, marked at a minimum with the location, subject, time and date of the photograph. A blank photo sign form is included in this document.

Recording Information:

Use a systematic method of recording information about each project, photo point, and photo. The following information should be entered on the photo-log forms (blank form included in this document) or in a dedicated notebook:

- Project or group name, and contract number (if applicable, e.g., for funded restoration projects)
- General location (stream, beach, city, etc.), and short narrative description of project's habitat type, goals, etc.
- Photographer and other team members
- Photo number
- Date
- Time (for each photograph)
- Photo point information, including:
    - Name or other unique identifier (abbreviated name and/or ID number)
    - Narrative description of location including proximity to and direction from notable landscape features like roads, fence lines, creeks, rock outcrops, large trees, buildings, previous photo points, etc. – sufficient for future photographers who have never visited the project to locate the photo point
    - Latitude, longitude, and altitude from map or GPS unit
- Magnetic compass bearing from the photo point to the subject
- Specific information about the subject of the photo
- Optional additional information: a true compass bearing (corrected for declination) from photo point to subject, time of sunrise and sunset (check newspaper or almanac), and cloud cover.

For ambient monitoring, the stream and shore walk form should be attached or referenced in the photo-log.

When monitoring the implementation of restoration, fuel reduction, or Best Management Practices (BMP) projects, include or attach to the photo-log a narrative description of observable progress in achieving the goals of the project. Provide supplementary information along with the photo, such as noticeable changes in habitat, wildlife, and water quality and quantity.

Archive all photos, along with the associated photo-log information, in a protected environment.

The Photo Point: Establishing Position of Photographer:

1. Have available a variety of methods for establishing position: maps, aerial photos, GPS, permanent markers and landmarks, etc. If the primary method fails (e.g., a GPS or lost

marker post) then have an alternate method (map, aerial photo, copy of an original photograph of the photo-point, etc).

2. Select an existing structure or landmark (mailbox, telephone pole, benchmark, large rock, etc.), identify its latitude and longitude, and choose (and record for future use) the permanent position of the photographer relative to that landmark. Alternatively, choose the procedure described in *Monitoring California's Annual Rangeland Vegetation* (UC/DANR Leaflet 21486, Dec. 1990). This procedure involves placing a permanently marked steel fence post to establish the position of the photographer.

3. For restoration, fuel reduction, and BMP projects, photograph the photo-points and carry copies of those photographs on subsequent field visits.

Determining the Compass Bearing:

1. Select and record the permanent magnetic bearing of the photo center view. You can also record the true compass bearing (corrected for declination) but do not substitute this for the magnetic bearing. Include a prominent landmark in a set position within the view. If possible, have an assistant stand at a fixed distance from both the photographer and the center of the view, holding a stadia rod if available, within the view of the camera; preferably position the stadia rod on one established, consistent side of the view for each photo (right or left side).
2. Alternatively, use the procedure described in *Monitoring California's Annual Rangeland Vegetation* (UC/DANR Leaflet 21486, Dec. 1990). This procedure involves placing a permanently marked steel fence post to establish the position of the focal point (photo center).
3. When performing ambient or event photo monitoring, and when a compass is not available, then refer to a map and record the approximate bearing as north, south, east or west.

Suggestions for Photo Points by Type of Project:

Ambient or Event Monitoring, Including Photography Associated with Narrative Visual Assessments:

1. When first beginning an ambient monitoring program take representative long and/or medium view photos of stream reaches and segments of shoreline being monitored. Show the positions of these photos on a map, preferably on the stream/shore walk form. Subjects to be photographed include a representative view of the stream or shore condition at the beginning and ending positions of the segment being monitored, storm drain outfalls, confluence of tributaries, structures (e.g., bridges, dams, pipelines, etc.).

2. If possible, take a close view photograph of the substrate (streambed), algae, or submerged aquatic vegetation.

3. Time series: Photographs of these subjects at the same photo points should be repeated annually during the same season or month if possible.

4. Event monitoring refers to any unusual or sporadic conditions encountered during a stream or shore walk, such as trash dumps, turbidity events, oil spills, etc. Photograph and record information on your photo-log and on your Stream and Shore Walk Visual Assessment form. Report pollution events to the Regional Board. Report trash dumps to local authorities.

All Restoration and Fuel Reduction Projects – Time Series:

Take photos immediately before and after construction, planting, or vegetation removal. Long term monitoring should allow for at least annual photography for a minimum of three years after the project, and thereafter at 5 years and ten years.

Meadow Restoration:

1. Aerial view (satellite or airplane photography) if available.

2. In the absence of an aerial view, a landscape, long view showing an overlapping sequence of photos illustrating a long reach of stream and meadow (satellite photos, or hill close by, fly-over, etc.)

3. Long view up or down the longitudinal dimension of the creek showing riparian vegetation growth bounded on each side by grasses, sedges, or whatever that is lower in height

4. Long view of conversion of sage and other upland species back to meadow vegetation

5. Long view and medium view of streambed changes (straightened back to meandering, sediment back to gravel, etc.)

6. Medium and close views of structures, plantings, etc. intended to induce these changes

Stream Restoration/stabilization:

1. Aerial view (satellite or airplane photography) if available.

2. In the absence of an aerial view, a landscape, long-view showing all or representative sections of the project (bluff, bridge, etc.)

3. Long view up or down the stream (from stream level) showing changes in the stream bank, vegetation, etc.

4. Long view and medium view of streambed changes (thalweg, gravel, meanders, etc.)

5. Medium and close views of structures, plantings, etc. intended to induce these changes.

6. Optional: Use a tape set perpendicular across the stream channel at fixed points and include this tape in your photos described in 3 and 4 above. For specific procedures refer to Harrelson, Cheryl C., C.L. Rawlins, and John P. Potyondy, *Stream Channel Reference Sites: An Illustrated Guide to Field Techniques*, United States Department of Agriculture, Forest Service, Rocky Mountain Forest and Range Experiment Station, General Technical Report RM-245.

Vegetation Management for Fire Prevention ("fuel reduction"):

1. Aerial view (satellite or airplane photography) if available.

2. In the absence of an aerial view, a landscape, long view showing all or representative sections of the project (bluff, bridge, etc.)

3. Long view (wide angle if possible) showing the project area or areas. Preferably these long views should be from an elevated vantage point.

4. Medium view photos showing examples of vegetation changes, and plantings if included in the project. It is recommended that a person (preferably holding a stadia rod) be included in the view for scale

5. To the extent possible include medium and long view photos that include adjacent stream channels.

Stream Sediment Load or Erosion Monitoring:

1. Long views from bridge or other elevated position.

2. Medium views of bars and banks, with a person (preferably holding a stadia rod) in view for scale.

3. Close views of streambed with ruler or other common object in the view for scale.

4. Time series: Photograph during the dry season (low flow) once per year or after a significant flood event when streambed is visible. The flood events may be episodic in the south and seasonal in the north.

5. Optional: Use a tape set perpendicular across the stream channel at fixed points and include this tape in your photos described in 1 and 2 above. For specific procedures refer to Harrelson, Cheryl C., C.L. Rawlins, and John P. Potyondy, *Stream Channel Reference Sites: An Illustrated Guide to Field Techniques*, United States Department of Agriculture, Forest Service, Rocky Mountain Forest and Range Experiment Station, General Technical Report RM-245.

PHOTO- LOG FORM

Project:

Location:

Date:

Photographer:

Team members:

Photo #	Time	Photo Point ID	Photo Pt. Description & Location	Bearing to Subject	Subject Description

General Notes or Comments (weather, cloud cover, time of sunrise and sunset, other pertinent information):

PHOTO SIGN FORM: Print this form on yellow paper. Complete the following information in black marker for each photograph. Include in the photographic view so that it will be legible in the finished photo.

Location:

Subject Description:

Date:

Time:

Handbook for Estimating Economic Benefits of Environmental Projects

For Inclusion in Benefit-Cost Assessments of Projects Proposed for funding Under California Propositions 84 and 1E

Prepared for the North Bay Watershed Association

December 2012

ECONorthwest
ECONOMICS • FINANCE • PLANNING

Cover photo credit: Flickr–Emerson Parcel Tidal Marsh, California, Pacific Southwest Region U.S. Fish and Wildlife Service

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Sarah Reich, Ann Hollingshead, and Ed MacMullan of ECONorthwest completed this project under contract to the North Bay Watershed Association. ECONorthwest is solely responsible for its content.

ECONorthwest is the oldest and most comprehensive economics consulting firm in the Pacific Northwest. We have a staff of 40 in offices in Portland and Eugene, Oregon and Boise, Idaho.

We gratefully acknowledge the assistance of the many individuals who provided us with information and insight, but emphasize that we, alone, are responsible for the contents of this handbook. We have compiled the information based on our general knowledge of environmental valuation methods, our review and application of the California Department of Water Resources' Guidelines and Proposal Solicitation Packages for Propositions 84 and 1E, information derived from government agencies, the reports of others, interviews of individuals, or other sources believed to be reliable. ECONorthwest has not independently verified the accuracy of information developed by others but cited here, and makes no representation regarding its veracity or completeness. Any statements nonfactual in nature constitute the authors' current opinions, which may change as more information becomes available

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TOMALES BAY
INTEGRATED COASTAL WATERSHED MANAGEMENT PLAN

Submitted To:
State of California
State Water Resources Control Board

Submitted By:
Tomales Bay Watershed Council
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Prepared Under a Grant From
The Water Security, Clean Drinking Water, Coastal and
Beach Protection Act of 2002

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July 2007

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LIST OF ACRONYMS AND ABBREVIATIONS USED IN THIS PLAN

ACR – Audubon Canyon Ranch, Cypress Grove Research Center
ASBS – Area of Special Biological Significance
AFY – Acre feet per year
Basin Plan – RWQCB Water Quality Control Plan
BCPUD – Bolinas Community Public Utility District
BMP – Best Management Practices
CCA – Critical Coastal Area
CCC – State of California Coastal Commission
CDHS – State of California Department of Health Services
CDFG - State of California Department of Fish and Game
Council – Tomales Bay Watershed Council
CWA – Clean Water Act
CWC – California Water Code
DWR – California Department of Water Resources
EAC – Environmental Action Committee of West Marin
EHS - Marin County Department of Environmental Health Services
EQIP – Environmental Quality Incentives Program
ESA – Endangered Species Act
GFNMS – Gulf of the Farallones National Marine Sanctuary
GGNRA – Golden Gate National Recreation Area
GIS – Geographic Information Systems
ICWMP – Integrated Coastal Watershed Management Plan
IRWM – Integrated Regional Watershed Management
IRWMP – Integrated Regional Watershed Management Plan
IPUD – Inverness Public Utility District
JARPA- Joint Aquatic Resources Permit Application
LWD – Large Woody Debris
MALT – Marin Agricultural Land Trust
MCDHHS – Marin County Department of Health and Human Services
MRCD – Marin Resource Conservation District
MCSTOPPP – Marin County Stormwater Pollution Prevention Program
MMWD – Marin Municipal Water District
NGO – Non-governmental organization
NOAA – National Oceanic and Atmospheric Administration
NMWD – North Marin Water District
NPS – Nonpoint source pollution
NRCS – Natural Resources Conservation Service
NSSP – National Shellfish Sanitation Program
OSDS – Onsite Sewage Disposal System
PRBO – Point Reyes Bird Observatory Conservation Science
PRNS - Point Reyes National Seashore
PRNSA - Point Reyes National Seashore Association
QA/QC – Quality assurance /Quality control

RWQCB - Regional Water Quality Control Board
SCWA – Sonoma County Water Agency
SFRWQCB - San Francisco Bay Regional Water Quality Control Board
SPAWN – Salmon Protection and Watershed Network
SWAMP – Surface Water Ambient Monitoring Program
SWQCB - State Water Quality Control Board
SWQPA – State Water Quality Protection Areas
TAC - Technical Advisory Committee
TBA- Tomales Bay Association
TBAG – Tomales Bay Agriculture Group
TBSTAC – Tomales Bay Shellfish Technical Advisory Committee
TBWC- Tomales Bay Watershed Council
TMDL – Total Maximum Daily Loads
TSS – Total Suspended Sediments
TU – Trout Unlimited
UCCE – University of California Cooperative Extension
UC –University of California
UCD – University of California at Davis
UNESCO – United Nations Educational, Scientific and Cultural Organization
USACE – U.S. Army Corps of Engineers
USCG – U.S. Coast Guard
USDA – U.S. Department of Agriculture
USDA-CREP – Conservation Reserve and Enhancement Program
USEPA – U.S. Environmental Protection Agency
USFWS – U.S. Fish and Wildlife Service
USGS – U.S. Geological Survey
USNPS – National Park Service
WDR – Waste Discharge Requirements
WMI – SWRCB Watershed Management Initiative
WMS – Waste Management Systems
WQS – Water Quality Standard

Riparian Revegetation Evaluation on California's North Coast Ranches

Final Report

Prepared by

University of California Cooperative Extension, County of Sonoma

June 2007

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Acknowledgements: We thank the California Coastal Conservancy, National Oceanographic and Atmospheric Administration's Restoration Center, and University of California Division of Agriculture and Natural Resources for the funding support to initiate and maintain this project. We would also like to thank the supportive and cooperative group of natural resource managers in the area who were forthcoming with potential project sites. Their willingness and contributions are truly the reason the project was possible. Specifically, we want to recognize the patience and assistance provided by Thomas Schott, Liza Prunuske, Paul Sheffer, Gale Ranch, Paul Martin, Nancy Scolari, Lisa Bush, Leah Mahan, Tresch Dairy, Robert Katz, Lauren Olin, Michael Hansen, Jim Nosera, and Sarah Nossaman. We also want to thank the following organizations that made significant contributions to this project.

- Marin Resource Conservation District (RCD)
- Mendocino County RCD
- Southern Sonoma RCD
- Gold Ridge RCD
- Natural Resources Conservation Service
- Prunuske Chatham, Inc.
- Bay Institute Students & Teachers Restoring A Watershed (STRAW)
- Bioengineering Associates
- Circuit Rider Productions, Inc.
- Casa Grande High School United Angler's Fish Hatchery
- Sonoma County Water Agency
- Ca. Department of Fish and Game Fort Ross Environmental Restoration
- Blue Mountain Center of Meditation
- Land and Places
- Forest, Soil & Water, Inc.
- City of Santa Rosa
- Sonoma State University
- Regional, State and National Parks
- U.S. Coast Guard

Cover photographs: *Coastal stream on ranch along Hwy. 1 in 1994 (upper) and in 2004 (lower). Photographs are courtesy of Marin Resource Conservation District.*

EXECUTIVE SUMMARY

Overview

We conducted a retrospective, cross-sectional survey of riparian revegetation projects on north coastal California working ranches. Our goal was to determine the efficacy of riparian restoration within the working landscape of California's rangelands. Specifically, we documented plant community succession and structure, and aquatic habitat response to restoration over time. This type of information is useful in confirming the benefits of restoration and directing improvements to restoration project design, implementation, and management that can improve the success of future projects.

In a typical revegetation project scenario, monitoring has focused on survival of planted vegetation and seldom extended beyond a contracted three to five year period. Rarely have monitoring surveys attempted to quantify the resulting available aquatic habitat and plant community structure over long-term time scales over multiple decades. Ecological restoration and riparian revegetation typically received minimal systematic project monitoring, evaluation, and feedback. The result has been limited documentation of project outcomes and effectiveness. Because of the considerable amount of riparian revegetation in north coastal California since the 1970's by private landowners, restoration practitioners, and financial and technical assistance agencies we were able to fill this data gap through our survey.

We surveyed 102 sites, totaling 19.4 kilometers, along streams in Marin, Mendocino, and Sonoma Counties from 2002 to 2005. Site selection focused on revegetation projects in mixed oak woodland tributaries with alluvial, gravel substrate reaches of minimal tree and shrub cover prior to project installation. Restoration methods at surveyed project sites included riparian revegetation, bioengineering bank stabilization, and herbivore management (removal, reduced numbers, or exclusionary fencing for livestock and/or deer). Surveyed sites included 89 riparian restoration sites and 13 non-restored sites which were near projects and representative of pre-project conditions.

We have documented riparian vegetation and aquatic habitat responses at temporal stages over a thirty-year period and compared the influences of common restoration methods on these responses. We have identified critical elements to guide site-specific potential for establishing native tree populations through both passive and active methods. Our restoration trajectory analysis correlated the abundance of nine tree genera to time since project installation, site

conditions and restoration treatment. Lastly, we developed practical guidelines and recommendations for monitoring riparian revegetation projects.

Findings and Discussion

Our results document significant improvements in both riparian vegetation and aquatic habitat metrics as the age of project sites increased. This confirmation of intended improvements to stream conditions should encourage the continuation of riparian revegetation projects. Our results also point to unintended outcomes resulting from such projects, such as increases in invasive plant species. This highlights the need to improve project design, implementation, and maintenance.

Outcomes from our models of individual tree genera response to restoration treatment method are useful for selecting implementation techniques. Generally, direct planting of the slower growing, late seral tree species significantly increases their abundance. By comparison, the presence of relict populations, perennial stream flow and floodplain area are more important in establishing early seral tree species.

Our results are useful for validating riparian restoration project success, improving future project designs, and guiding how to efficiently monitor program effectiveness. We recommend project planning continue to follow site-specific approaches and our results provide insight for this process. Concepts such as the relative bank height or elevation above the stream channel, stream flow regime, identification of relict seed sources of early seral tree species and soil percent clay are factors that restoration practitioners and funders should consider when adapting project designs to site-specific conditions and species-specific objectives.

In addition to providing restorationists, landowners, and funders with useful direction to improve the success of riparian revegetation projects, we have developed a document for conducting monitoring of such projects. This document has two elements: 1) Developing a monitoring program; and 2) Observations and recommendations for monitoring methodologies.

Natural resource managers and restoration practitioners should be able to use these results to set realistic and quantified project objectives for both riparian vegetation and aquatic habitat metrics. They should also be useful in site design and implementation decision making, as well as site management, including the use of efficient monitoring. Armed with this understanding, the restoration partnership between the landowner, practitioner, and funder will increase their

efficiency and effectiveness, resulting in wiser uses of resources to implement even more successful stream revegetation projects.

The principal findings, considerations, and recommendations of our study are:

- Stream and river revegetation has resulted in improvements to aquatic habitat and riparian vegetation community and structure at the surveyed sites. This confirmation of long-term intended and beneficial outcomes supports the continuation of these practices to improve watershed functions.
- Passive restoration methods (controlling herbivory using livestock and/or deer exclosures, livestock removal, or managing livestock by reducing their intensity) and active revegetation techniques (planting and/or bioengineering) are both effective at bringing about beneficial responses in aquatic habitat and plant community structure at restored project sites. Active methods accelerate this response for metrics such as canopy cover and bank stability in the first ten years after project implementation. In general, the magnitude of the response from both methods for numerous metrics converges after approximately 10 to 15 years. The inference is that there is a limit, or ceiling, to rate of response and ending value for studied metrics over time regardless of the restoration method employed.
- The one notable exception or difference between active and passive revegetation methods is the increase in tree species diversity achieved consistently through time with direct planting methods.
- Both active and passive revegetation methods are viable tools for the restoration partnership to use. Selection of restoration techniques should be based upon a balance between site-specific objectives, programmatic goals and resource allocation. An accelerated rapid response may be desired and can be achieved through active methods, but with an associated higher project budget. Alternatively, a program may place a premium on treating the greatest length of stream per restoration dollar spent, which favors the use of passive methods and requires a longer time horizon to achieve project site response. Certain sites require active methods to address acute bank stability issues, provide a seed source, or develop an active floodplain that contribute to the establishment and continued propagation of riparian vegetation. Our general recommendation is that project design should be guided by site-specific potential for passive revegetation and active methods should be used to enhance that potential.

- Early seral, fast growing tree species recovery is accelerated in the presence of relict populations at or near the site. Project design that accounts for the presence and location of seed source for either desired or invasive species will be able to capitalize upon or prevent their influence on project site response.
- Planting is generally required for the recovery of late seral, slower growing tree species within 30 years. Furthermore, survival and establishment is related to soil texture - oaks (*Quercus sp.*) and Douglas-fir (*Pseudotsuga menziesii*) have greater densities in high percentage clay soils and California Buckeye (*Aesculus californica*) having greater densities in low percentage clay soils.
- Resulting riparian forest composition and structure is related to geomorphology at project sites. Accordingly, landform distribution should be used in decisions regarding where to plant which species.
- There is a consistent transition in the understory plant community from annual to perennial herbs to shrubs over time. This transition appears to culminate in sites dominated by shrub and vine species, with Himalayan blackberry (*Rubus discolor*) as the most common component. There is also a corresponding decrease in sedges (*Carex sp.*) and rushes (*Juncus sp.*).
- The change in understory plant community composition points to the need for vegetation management at project sites, including the identification of appropriate weed control methods and a funding process to support this long-term project maintenance task.
- Recommendations for improving project design and implementation include:
    - Delay planting for one to two years post project site fencing, to learn where and what plant species will colonize through natural regeneration. The potential for natural regeneration, and thus this recommendation, to be effective is greatest at sites with floodplain access, relict species, and perennial stream flow.
    - Planting more tree willow species such as red and/or shining willow where appropriate.
    - Early seral species will have to be planted if they are desired at the site above frequently flooded locations. Below that flood stage, project design can seek to capitalize on the sites natural regeneration potential.
    - Where streambanks are unstable, bioengineering and other bank stabilization methods are needed to facilitate streambank revegetation.

- Qualitative implementation and effectiveness monitoring with photo-point methods is useful to document site response following project implementation and should be continued.
- Quantitative effectiveness monitoring is needed for a select number of site response metrics - canopy cover, width-to-depth ratio, maximum pool depth, and tree/ shrub composition by cover. Baseline values for these metrics can be collected during the project implementation phase if the appropriate expertise and technical support needed to conduct the monitoring is available.
- Future monitoring visits to project sites should be timed appropriately to guide decisions about adaptive management and strategic intervention. For holistic plant community management and controlling invasive, non-native species, project sites should be evaluated at approximately five to ten years post project implementation.

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NATURAL RESOURCES CONSERVATION SERVICE

CONSERVATION PRACTICE STANDARD

CRITICAL AREA PLANTING

(Ac.)

CODE 342

DEFINITION

Establishing permanent vegetation on sites that have, or are expected to have, high erosion rates, and on sites that have physical, chemical or biological conditions that prevent the establishment of vegetation with normal practices.

PURPOSE

Stabilize stream and channel banks, and shorelines.

Stabilize areas with existing or expected high rates of soil erosion by wind or water.

Rehabilitate and revegetate degraded sites that cannot be stabilized using normal establishment techniques.

Stabilize coastal areas, such as sand dunes and riparian areas.

CONDITIONS WHERE PRACTICE APPLIES

This practice applies to highly disturbed areas such as:

- active or abandoned mined lands;
- urban conservation sites;
- road construction areas;
- conservation practice construction sites;
- areas needing stabilization before or after natural disasters such as floods, hurricanes, tornados and wildfires;
- eroded banks of natural channels, banks of newly constructed channels, and lake shorelines;
- other areas degraded by human activities or natural events.

CRITERIA

General Criteria Applicable To All Purposes

Site Preparation. A site investigation shall be conducted to identify any physical, chemical, or biological conditions that could affect the successful establishment of vegetation.

Areas to be planted will be cleared of unwanted materials and smoothed or shaped, if needed, to meet planting and landscaping purposes.

A suitable seedbed shall be prepared for all seeded species. Compacted layers will be ripped and the soil re-firmed prior to seedbed preparation.

Species Selection. Species selected for seeding or planting shall be suited to current site conditions and intended uses, and be resistant to diseases or insects common to the site or location.

Selected species will have the capacity to achieve adequate density and vigor to stabilize the site within an appropriate period.

No plants on the Federal or state noxious weeds list shall be planted.

Establishment of Vegetation. Seeds will be planted using the method or methods best suited to site and soil conditions.

Sod placement shall be limited to areas that can naturally supply needed moisture or sites that can be irrigated during the establishment period.

Sod will be placed and anchored using techniques to ensure that it remains in place until established.

Species, rates of seeding or planting, minimum quality of planting stock (e.g. pure live seed

(PLS) or stem caliper), method of seedbed preparation, and method of establishment shall be specified before application. Only viable, high quality seed or planting stock will be used.

Seeding or planting shall be done at a time and in a manner that best ensures establishment and growth of the selected species. What constitutes successful establishment (e.g. minimum percent ground/canopy cover, percent survival, stand density) shall be specified before application.

Planting shall be done during approved times for the species to be used.

Apply soil amendments (e.g. lime, fertilizer, compost) according to the requirements in the local Field Office Technical Guide.

Plantings shall be mulched as necessary to ensure establishment. Other disturbed areas shall be mulched as necessary to prevent erosion.

Additional Criteria to Stabilize Stream and Channel Banks and Shorelines

When slopes are modified for seeding, topsoil will be stockpiled and spread over areas to be planted as needed to meet planting and landscaping needs.

Bank and Channel Slopes. Channel side slopes shall be shaped so that they are stable and allow establishment and maintenance of desired vegetation.

Slopes steeper than 2:1 shall not be stabilized using vegetation alone. A combination of vegetative and structural measures will be used on these slopes to ensure adequate stability.

Species Selection. Plant material used for this purpose shall:

- adapted to the hydrologic zone (see Fig. 1) into which they will be planted.
- be adapted and proven in the regions in which they will be used.
- when mature, produce plant communities that are compatible with those in the area.
- protect the channel banks but not restrict channel capacity.

Establishment of Vegetation. The species used, planting rates, spacing, and methods and dates of planting shall be based on plant materials program trials or other technical guidance, such as local planting guides or technical notes.

Identify, mark, and protect desirable existing vegetation during practice installation.

A combination of vegetative and structural measures using living and inert material shall be used when flow velocities, soils, and bank stability preclude stabilization by vegetative establishment alone.

If the existing vegetation on a site will compete with species to be established vegetatively (e.g. bare-root, containerized, ball-and-burlap, potted), it will be controlled in a manner that ensures the successful establishment of the planted species.

Site Protection and Access Control. Grazing animal access to planted areas will be controlled for a minimum of two growing seasons during the establishment period.

All areas to be grazed will have a grazing plan that meets the criteria in the local Field Office Technical Guide.

Grazing shall be permanently excluded on high hazard sites, such as cut banks, areas of seepage or other potentially unstable areas.

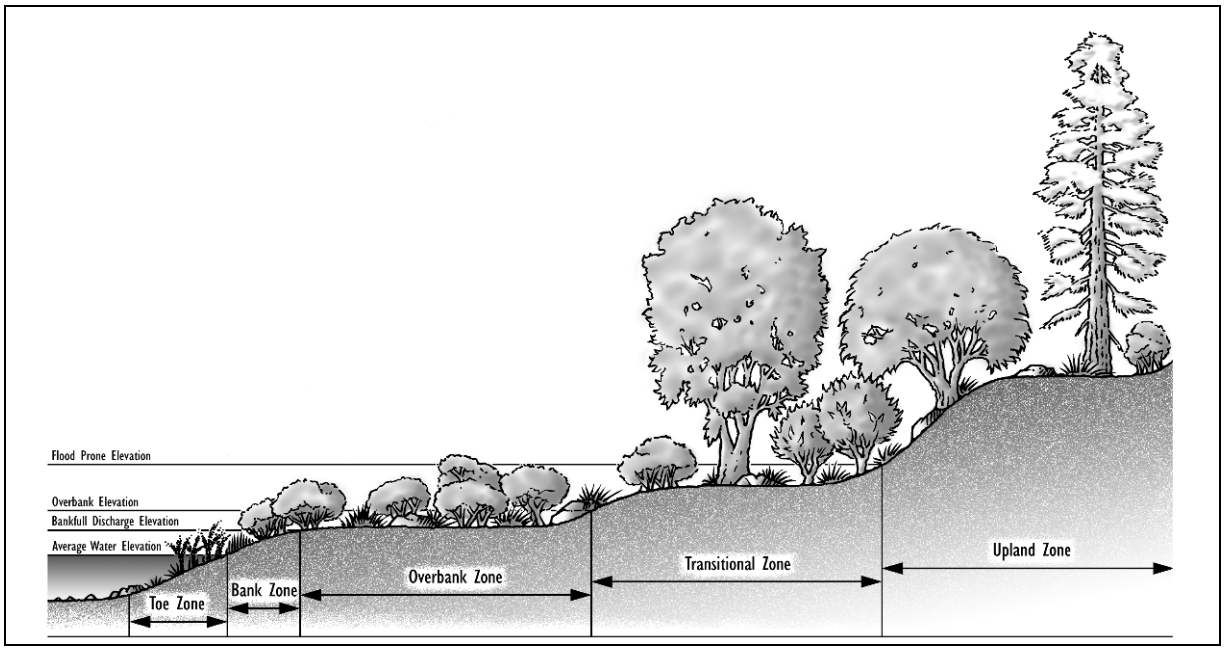


Figure 1. Location of hydrologic zones along a channel or shoreline.

Definitions and descriptions of hydrologic zones used for channels and shorelines:

Bankfull Discharge Elevation - In natural streams, it is the elevation at which water fills the channel without overflowing onto the flood plain.

Bank Zone - The area above the Toe Zone located between the average water level and the bankfull discharge elevation. Vegetation may be herbaceous or woody, and is characterized by flexible stems and rhizomatous root systems.

Overbank Zone - The area located above the bankfull discharge elevation continuing upslope to an elevation equal to two thirds of the flood prone depth. Vegetation is generally small to medium shrub species.

Toe Zone - The portion of the bank that is between the average water level and the bottom of the channel, at the toe of the bank. Vegetation is generally herbaceous emergent aquatic species, tolerant of long periods of inundation.

Transitional Zone - The area located between the overbank zone, and the flood prone width elevation. Vegetation is usually larger shrub and tree species.

Upland Zone – The area above the Transitional Zone; this area is seldom if ever saturated.

Note: some channels or shorelines have fewer than four hydrologic zones because of differences in soils, topography, entrenchment and/or moisture regime.

Additional Criteria to Rehabilitate and Revegetate Degraded Sites that Cannot Be Stabilized through Normal Farming Practices.

If gullies or deep rills are present, they will be filled and leveled as necessary to allow equipment operation and ensure proper site and seedbed preparation.

Based on a soil test and other appropriate site evaluations, soil amendments will be added as necessary to ameliorate or eliminate physical or chemical conditions that inhibit plant establishment and growth.

Additional Criteria to Restore Coastal Areas, such as Sand Dunes and Riparian Areas

Plants for sand dunes and coastal sites must be able to survive being buried by blowing sand, sand blasting, salt spray, salt water flooding, drought, heat, and low nutrient supply.

Local plant lists including appropriate species shall be developed and utilized.

Sand trapping devices such as sand fences or brush matting shall be included in the revegetation/ stabilization plans where applicable.

CONSIDERATIONS

Species or mixes that are adapted to the site and have multiple values should be considered. Native species may be used when appropriate for the site.

To benefit pollinators and other wildlife, flowering shrubs and wildflowers with tough root systems and good soil holding capacity also should be considered for incorporation as a small percentage of a larger grass-dominated planting. Where appropriate consider a diverse mixture of legumes and forbs to support pollinator habitat.

Avoid species that may harbor pests. Species diversity should be considered to avoid loss of function due to species-specific pests.

Planning and installation of other conservation practices such as Diversion (code 362), Obstruction Removal (code 500), Subsurface Drain (code 606), or Underground Outlet (code 620) may be necessary to prepare the area or ensure vegetative establishment.

Areas of vegetation established with this practice can create habitat for various type of wildlife. Maintenance activities, such as mowing or spraying, can have detrimental effects on certain species. Perform management activities at the times and in a manner that causes the least disruption to wildlife.

PLANS AND SPECIFICATIONS

Prepare plans and specifications for each field or management unit according to the criteria and operation and maintenance sections of this standard. Record practice specifications using approved specification sheets, job sheets or other acceptable documentation.

The following elements shall be addressed in the plan, as applicable, to meet the intended purpose.

- Site preparation
- Topsoil requirements
- Fertilizer application
- Seedbed/planting area preparation
- Methods of seeding/planting
- Time of seeding/planting
- Selection of species
- Seed/plant source
- Seed analysis
- Seeding rate/plant spacing
- Mulching
- Supplemental water needed for establishment
- Protection of plantings

OPERATION AND MAINTENANCE

Use of the area shall be managed as long as necessary to ensure the site remains stabile.

Plantings shall be protected from pests (e.g. weeds, insects, diseases, livestock, or wildlife) as necessary to ensure long-term survival.

Inspections, reseeding or replanting, and fertilization may be needed to ensure that this practice functions as intended throughout its expected life. Observation of establishment progress and success should be performed at regular intervals until the practice has met the criteria for successful establishment and implementation.

REFERENCES

Federal Interagency Stream Restoration Working Group. 1998. Stream corridor restoration: principles, processes, and practices. National Engineering Handbook, Part 653.

USDA-NRCS. 2007. National Engineering Handbook, Part 654. Stream restoration guide.

USDA-NRCS. 2010. The PLANTS Database (http://plants.usda.gov, checked September 2010). National Plant Data Center.

NATURAL RESOURCES CONSERVATION SERVICE

CONSERVATION PRACTICE STANDARD

STREAMBANK AND SHORELINE PROTECTION

(Ft.)

CODE 580

DEFINITION

Treatment(s) used to stabilize and protect banks of streams or constructed channels, and shorelines of lakes, reservoirs, or estuaries.

PURPOSE

- To prevent the loss of land or damage to land uses, or facilities adjacent to the banks of streams or constructed channels, shoreline of lakes, reservoirs, or estuaries including the protection of known historical, archeological, and traditional cultural properties.
- To maintain the flow capacity of streams or channels.
- Reduce the offsite or downstream effects of sediment resulting from bank erosion.
- To improve or enhance the stream corridor for fish and wildlife habitat, aesthetics, recreation.

CONDITIONS WHERE PRACTICE APPLIES

This practice applies to streambanks of natural or constructed channels and shorelines of lakes, reservoirs, or estuaries where they are susceptible to erosion. It does not apply to erosion problems on main ocean fronts, beaches or similar areas of complexity.

CRITERIA

General Criteria Applicable to All Purposes

Treatments shall be in accordance with all applicable local, state and federal laws and regulations.

Treatments applied shall seek to avoid adverse effects to endangered, threatened, and candidate species and their habitats, whenever possible.

Treatments applied shall seek to avoid adverse effects to archaeological, historic, structural, and traditional cultural properties, whenever possible.

An assessment of unstable streambank or shoreline sites shall be conducted in sufficient detail to identify the causes contributing to the instability (e.g. livestock access, watershed alterations resulting in significant modifications of discharge or sediment production, in channel modifications such as gravel mining, head cutting, water level fluctuations, boat-generated waves, etc.).

Proposed protective treatments to be applied shall be compatible with improvements being planned or installed by others.

Protective treatments shall be compatible with the bank or shoreline materials, water chemistry, channel or lake hydraulics, and slope characteristics above and below the water line.

End sections of treatment areas shall be adequately anchored to existing treatments, terminate in stable areas, or be otherwise stabilized to prevent flanking of the treatment.

Protective treatments shall be installed that result in stable slopes. Design limitations of the bank or shoreline materials and type of measure installed shall determine steepest permissible slopes.

Designs will provide for protection of installed treatments from overbank flows resulting from upslope runoff and flood return flows.

Internal drainage for bank seepage shall be provided when needed. Geotextiles or properly

Conservation practice standards are reviewed periodically, and updated if needed. To obtain the current version of this standard, contact the Natural Resources Conservation Service State Office, or visit the Field Office Technical Guide.

designed filter bedding shall be incorporated with structural measures where there is the potential for migration of material from behind the measure.

Treatments shall be designed to account for any anticipated ice action, wave action, and fluctuating water levels.

All disturbed areas around protective treatments shall be protected from erosion. Disturbed areas that are not to be cultivated shall be protected as soon as practical after construction.

Vegetation shall be selected that is best suited for the site conditions and achieves the intended purpose(s).

In order to ensure plant community establishment and integrity, a vegetative management plan shall be prepared in accordance with NRCS conservation practice standard Critical Area Planting, Code 342.

Additional Criteria for Streambanks

Stream segments to be protected shall be classified according to a system deemed appropriate by the state. Segments that are incised or that contain the 5-year return period (20 percent probability) or greater flows shall be evaluated for further degradation or aggradation.

A site assessment shall be performed to determine if the causes of instability are local (e.g. poor soils, high water table in banks, alignment, obstructions deflecting flows into bank, etc.) or systemic in nature (e.g. aggradation due to increased sediment from the watershed, increased runoff due to urban development in the watershed, degradation due to channel modifications, etc.). The assessment need only be of the extent and detail necessary to provide a basis for design of the bank treatments and reasonable confidence that the treatments will perform adequately for the design life of the measure.

Changes in channel alignment shall not be made without an assessment of both upstream and downstream fluvial geomorphology that evaluates the affects of the proposed alignment. The current and future discharge-sediment regime shall be based on an assessment of the watershed above the proposed channel alignment.

Bank protection treatment shall not be installed in channel systems undergoing rapid and extensive changes in bottom grade and/or alignment unless the treatments are designed to control or accommodate the changes. Bank treatment shall be constructed to a depth at or below the anticipated lowest depth of streambed scour.

If the failure mechanism is a result of the degradation or removal of riparian vegetation, stream corridor restoration shall be implemented, where feasible, (see Additional Criteria for Stream Corridor Improvement) as well as treating the banks.

Toe erosion shall be stabilized by treatments that redirect the stream flow away from the toe or by structural treatments that armor the toe. Additional design guidance is found in the EFH Part 650, Chapter 16, Streambank and Shoreline Protection.

Where toe protection alone is inadequate to stabilize the bank, the upper bank shall be shaped to a stable slope and vegetated, or shall be stabilized with structural or soil-bioengineering treatments.

Channel clearing to remove stumps, fallen trees, debris, and sediment bars shall only be performed when they are causing or could cause unacceptable bank erosion, flow restriction, or damage to structures. Habitat forming elements that provide cover, food, pools, and water turbulence shall be retained or replaced to the extent possible.

Treatments shall be functional and stable for the design flow and sustainable for higher flow conditions.

Treatments shall not induce an increase in natural erosion.

Treatments shall not limit stream flow access to the floodplain.

Where flooding is a concern, the effects of protective treatments shall not increase flow levels above those that existed prior to installation.

Additional Criteria for Shorelines

All revetments, bulkheads or groins are to be no higher than 3 feet (1 meter) above mean high tide, or mean high water in non-tidal areas

Structural shoreline protective treatments shall be keyed to a depth to prevent scour during low water.

For the design of structural treatments, the site characteristics below the waterline shall be evaluated for a minimum of 50 feet (15 meters) horizontal distance from the shoreline measured at the design water surface.

The height of the protection shall be based on the design water surface plus the computed wave height and freeboard. The design water surface in tidal areas shall be mean high tide.

When vegetation is selected as the protective treatment, a temporary breakwater shall be used during establishment when wave run up would damage the vegetation.

Additional Criteria for Stream Corridor Improvement

Stream corridor vegetative components shall be established as necessary for ecosystem functioning and stability. The appropriate composition of vegetative components is a key element in preventing excess long-term channel migration in re-established stream corridors. The establishment of vegetation on channel banks and associated areas shall also be in accordance with conservation practice standard Critical Area Planting, Code 342.

Treatments shall be designed to achieve habitat and population objectives for fish and wildlife species or communities of concern as determined by a site-specific assessment or management plan. Objectives shall be based on the survival and reproductive needs of populations and communities, which include habitat diversity, habitat linkages, daily and seasonal habitat ranges, limiting factors and native plant communities. The type, amount, and distribution of vegetation shall be based on the requirements of the fish and wildlife species or communities of concern to the extent possible.

Treatments shall be designed to meet aesthetic objectives as determined by a site-specific assessment or management plan. Aesthetic objectives shall be based on human needs, including visual quality, noise control, and microclimate control. Construction materials, grading practices, and other site development elements shall be selected and designed to be compatible with adjacent land uses.

Treatments shall be designed to achieve recreation objectives as determined by a site-specific assessment or management plan. Safety requirements shall be based on type of human use and recreation objectives.

CONSIDERATIONS

When designing protective treatments, consider should be given to the changes that may occur in the watershed hydrology and sedimentation over the design life of the treatments.

Consider utilizing debris removed from the channel or streambank into the treatment design when it is compatible with the intended purpose to improve benefits for fish, wildlife and aquatic systems.

Use construction materials, grading practices, vegetation, and other site development elements that minimize visual impacts and maintain or complement existing landscape uses such as pedestrian paths, climate controls, buffers, etc. Avoid excessive disturbance and compaction of the site during installation.

Utilize vegetative species that are native and/or compatible with local ecosystems. Avoid introduced, invasive, noxious or exotic species that could become nuisances. Consider species that have multiple values such as those suited for biomass, nuts, fruit, browse, nesting, aesthetics and tolerance to locally used herbicides. Avoid species that may be alternate hosts to disease or undesirable pests. Species diversity should be considered to avoid loss of function due to species-specific pests. Species on noxious plant lists should not be used.

Select plant materials that provide habitat requirements for desirable wildlife and pollinators. The addition of native forbs and legumes to grass mixes will increase the value of plantings for both wildlife and pollinators.

Treatments that promote beneficial sediment deposition and the filtering of sediment, sediment-attached, and dissolved substances should be considered.

Consider maintaining or improving the habitat value for fish and wildlife by including treatments that provide aquatic habitat in the treatment

design and that may lower or moderate water temperature and improve water quality.

Consider the need to stabilize side channel inlets and outlets and outlets of tributary streams from erosion.

Consider aquatic habitat when selecting the type of toe stabilization.

Consider maximizing adjacent wetland functions and values with the project design and minimize adverse effects to existing wetland functions and values.

Livestock exclusion shall be considered during establishment of vegetative treatments and appropriate grazing practices applied after establishment to maintain plant community integrity. Wildlife may also need to be controlled during establishment of vegetative treatments. Temporary and local population control methods should be used with caution and within state and local regulations.

When appropriate, establish a buffer strip and/or diversion at the top of the bank or shoreline protection zone to help maintain and protect installed treatments, improve their function, filter out sediments, nutrients, and pollutants from runoff, and provide additional wildlife habitat.

Consider conservation and stabilization of archeological, historic, structural and traditional cultural properties when applicable.

Consider safety hazards to boaters, swimmers, or people using the shoreline or streambank when designing treatments.

Protective treatments should be self-sustaining or require minimum maintenance.

PLANS AND SPECIFICATIONS

Plans and specifications for streambank and shoreline protection shall be prepared for specific field sites and based on this standard and shall describe the requirements for applying the practice to achieve its intended purpose. Plans shall include treatments to minimize erosion and sediment production during construction and provisions necessary to comply with conditions of any environmental agreements, biological opinions or other terms of applicable permits.

OPERATION AND MAINTENANCE

An operation and maintenance plan shall be prepared for use by the owner or others responsible for operating and maintaining the system. The plan shall provide specific instructions for operating and maintaining the system to insure that it functions properly. It shall also provide for periodic inspections and prompt repair or replacement of damaged components or erosion.

REFERENCES

NEH Part 650, Chapter 16, Streambank and Shoreline Protection..

Pathfinder International

Guidance for Global Leaders

"Valuation of STRAW"
A Project of the Bay Institute
May 2007

QuickTime™ and a
TIFF (Uncompressed) decompressor
are needed to see this picture.

<u>An Independent Evaluation by Pathfinder International Consulting</u>
Catriona MacGregor Glazebrook, J.D., M.S., R.M.A.

<u>Purpose of this Report</u>

This report reflects a comprehensive and independent "valuation" of the STRAW (Students and Teachers Restoring a Wetland) project of the Bay Institute. This evaluation

was undertaken with a group of peers and environmental education experts (cumulative EE experience of over 90 years) and reflects an extensive base of knowledge in the field.

In documenting the experience of key STRAW contacts and producing a report, STRAW's leadership hoped to show staff, supporters and the community the program's value and strengths. Additionally, by understanding the program's strengths as described in this report, STRAW's leaders can build upon those strengths in the future.

Overview of STRAW Project of the Bay Institute,

The mission of the Bay Institute is: *To protect and restore the ecosystems of San Francisco Bay, the Sacramento-San Joaquin Delta, and the rivers, streams, and watersheds tributary to the Estuary.*

The Bay Institute was founded in 1981. The Bay Institute uses a combination of scientific research, political advocacy, and public education to work toward the environmental restoration of the entire watershed, which drains, into San Francisco Bay. This watershed includes the Sacramento River and the San Joaquin Rivers as well as their tributaries, Suisan Marsh, San Pablo Bay, and San Francisco Bay. The land area covers 40 percent of California. Nearly half of the surface water in California starts as rain or snow that falls in this area, and about half of that is diverted for use on farms, in homes, and in factories.

The STRAW Project is One Project of the Bay Institute.

STRAW promotes an educational approach known as "project-based learning," in which students are encouraged to pursue their own questions and projects focused on real-world issues. Through linking their work with larger community concerns, students develop independent and critical thinking, investigative and social skills, and a willingness to experiment. They also apply and deepen their knowledge of academic subjects through exploring creek ecology and hydrology, bird and aquatic insect studies, water quality monitoring, mapping, native plants, and nature writing and art with the assistance of STRAW staff and partners.

Since 1993, more than 12,000 students have participated in over 200 STRAW restorations on rural and urban creeks, planting over 23,000 native plants and restoring approximately 79,000 linear feet of creek banks or almost 66 acres.

As a project of The Bay Institute and participating partners, STRAW coordinates and sustains a network of teachers, students, restoration specialists and other community members as it plans and implements watershed studies and restoration projects in Marin and Sonoma counties. Its project-based learning approach supports its goals of empowering students, supporting teachers, restoring the environment and reconnecting communities.

In addition to its restoration projects, STRAW offers an annual three-day teacher training institute known as "Watershed Week." Held before the school year begins, Watershed Week attracts between 80 and 100 participants in the STRAW network, including teachers, partners and other education providers, to study local ecology, learn hands-on methods of scientific inquiry, share ideas about integrating environmental fieldwork and prepare for creek restoration activities in the coming year.

STRAW grew out of the work of the Shrimp Project, begun in 1992 by a fourth-grade class at Brookside School in Marin County. The class wanted to do something about the problem of endangered species and chose to focus on the plight of the endangered California freshwater shrimp. The class pioneered methods for students to conduct professional restoration of riparian corridors, and word spread about the Shrimp Project's accomplishments.

Valuation Methodology

To best understand the program's strengths, and to receive the most valuable information from STRAW participants - a method known as Appreciative Inquiry was used. Appreciative Inquiry is about the co-evolutionary search for the best in people, their organizations, and the relevant world around them.

AI involves, in a central way, the art and practice of asking questions that strengthen a system's capacity to apprehend, anticipate, and heighten positive potential. AI can be used in all types of organizational processes including strategic planning, team building and evaluation methodology. This is one of the few evaluation methodologies in existence that routinely results in strengthening organizations and programs in its wake. The valuation is the first step in a four-step process (see Appendix) but can also stand on its own for evaluation purposes.

The STRAW project has accumulated significant quantitative data pointing to its value including the large number of participants and the large number of watershed acres restored. In addition, the technical merit of the restoration work has been been found to be of excellent quality according to a professional restoration agency. Thus the emphasis throughout this process has been to collect in-depth qualitative data that speaks to a three-tiered level of inquiry and by focusing on the greatest value of environmental education to society and to participants.

This qualitative data can be objectively measured especially when it is compared to qualitative data from a large number of similar programs and organizations. Consequently, conclusions are supported by existing data gathered from over one hundred environmental organizations in the United States and Internationally.

In addition, unlike the information that is often obtained from traditional evaluation methods using simple written surveys and quantitative number rating systems, AI involves gathering information and data through the an in-depth process of one on one

interviews. This process provides much greater understanding about the program and about the benefits to program participants. Thus, information and stories about the meaningfulness of the program to participants and to society can be gathered and understood. This level of inquiry incorporates then – not just "superficial" statistical information but provides the more in-depth information than even a focus group process routinely used in traditional valuation methodology with an emphasis on thoroughness.

The STRAW valuation includes obtaining in depth feedback from STRAW staff members, STRAW participants (both students and teachers) and peers in the environmental education community. The peers, members of SEEC, attended a restoration day of the STRAW program. One peer also attended a teacher training held by STRAW.

Note that this particular "valuation" places a high level of focus on peer assessment. In general, "valuations" that include peers, who are experts in the field, tend to reflect a better understanding of the value and quality of the program being reviewed in light of the spectrum of environmental education programs that are offered.

The peers in the environmental education field that participated in this "valuation" developed the following three overriding levels of inquiry as the three most valuable aspects of any environmental education program to society and participants.

THE THREE TIERED LEVEL OF INQUIRY:

- *How does STRAW's program empower people to make a change?*
- *How does STRAW's program create a sense of wonder?*
- *How does STRAW's program inform and revitalize people's relationship to nature and their community?*

Summary of Findings

After having conducted valuations and assessments of over one hundred environmental programs in the United States and Internationally, the STRAW project of the Bay Institute stands out as an exceptional program, and in the top ten percent of environmental education programs reviewed.

This conclusion is based on the following:

I. *The successful implementation of the organizations' mission and goals, around leadership, community building, environmental education delivery and restoration of watershed habitat.* Although the STRAW staff requested that this evaluation focus primarily on environmental education versus restoration results to some extent these two elements cannot be separated by the very fact that the high quality of the educational experience is directly linked to the "real work" that is being done through the stream bank restorations. In addition, restoring stream banks and riparian habitat is a proven and highly effective way to address growing eco-system degradation and to restore the environment. Restoration beyond simply conservation makes actual tangible improvements to the environment.

Thus, one of the greatest strengths of the STRAW project is that it is about "real work". Students and teachers, along with their parents and STRAW partners ARE restoring a watershed. This singular focus brings a sense of importance to the project and the work being accomplished. People generally tend to shift into a high level of engagement when they know that their work matters. This imbues the project participants with a greater sense of commitment to their work and to each other.

In addition, and what makes this project remarkable is that the "real work' and professional quality restoration work is being completed, in part, by students.

II. *Overall, STRAW excels in the three areas deemed by environmental education experts and peers as the most valuable elements of any environmental education program.* If these three areas were placed in order of their highest level of delivery – the STRAW project equally empowers people to make a change while informing and revitalizing people's relationship to nature and their community. Following close behind is the STRAW projects ability to create a sense of wonder for the participants of its program.

III. *The STRAW program is an excellent example of an empowering leadership model.* The Leadership of the Bay Institute (Executive Director Grant Davis) and the STRAW staff and founders (especially Laurette Rogers and Sandy Neumann) have worked to engender a working environment of respect, trust and mutually shared ownership among staff, partners and participants. The Bay Institute may want to consider offering the STRAW project as a management model for other organizations to follow.

IV. *In addition to the high quality of the environmental education experience that STRAW provides, STRAW is also a highly regarded community catalyst that brings together students, teachers, ranchers and restoration professionals around shared goals.* Students not only participate in restoring a watershed but they see how the work that they do benefits the ranch owners land and recognize that they are helping central members of the community. In addition, ranchers are directly experiencing the benefits of an environmental education program by having the STRAW team come to their land and improve it.

V. *Meeting the needs of public school teachers has been a growing challenge in the field of environmental education. STRAW has been highly successful in providing an experience that is supportive and inspiring to teachers and the needs of their students.* STRAW has been able to maintain long-term relationships with their teachers many of whom return year after year.

VI. *Along with long-term improvements to the environment by restoring habitat, the STRAW project also has had significant long-term positive impacts on the participants and "beneficiaries" of the program.* This is a measurable impact in part since the STRAW project has been operating for nine years. Some of the measurable benefits to program participants include:

- Improvements to ranch owners' land;
- Career enhancing catalyst for teachers;
- Empowering and informative experience for young people;
- Affirming work experience for STRAW staff.

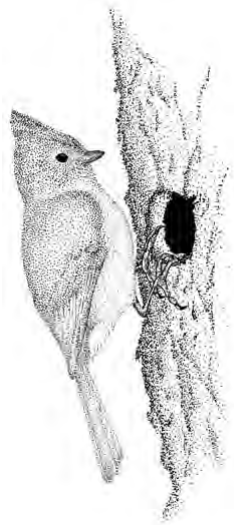
These will be discussed in more detail in the narrative section.

Bringing the Birds Back:

A Guide to Habitat Enhancement in Riparian and Oak Woodlands for the North Bay Region

Kim Kreitinger and Tom Gardali

California Partners in Flight
PRBO Conservation Science



Recommended Citation:
Kreitinger, K. and T. Gardali. 2006. Bringing the Birds Back: A Guide to Habitat Enhancement in Riparian and Oak Woodlands for the North Bay Region. California Partners in Flight Regional Bird Conservation Plan No. 1, http://www.prbo.org/calpif.

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Lark Sparrow (*Chondestes grammacus*)

For additional copies of this guide, please contact PRBO Conservation Science at (415) 868-0655 or visit the California Partners in Flight web site at: http://www.prbo.org/calpif

Marin County
Watershed Management Plan
Administrative Draft

April 2004

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GROUNDWORK

A HANDBOOK FOR SMALL-SCALE EROSION CONTROL IN COASTAL CALIFORNIA

GROUNDWORK:

A HANDBOOK FOR SMALL-SCALE EROSION CONTROL IN COASTAL CALIFORNIA

2nd Edition

Marin Resource Conservation District
Marin County Stormwater Pollution Prevention Program

Groundwork was first written in 1987 through a grant from the State Coastal Conservancy to the Marin Resource Conservation District. This revision has been largely funded by Marin County Stormwater Pollution Prevention Program and a grant from the State Water Resources Control Board to the Marin County Department of Public Works. Additional funds were generously provided by the Marin Community Foundation, North Coast Resource Conservation and Development Council, and from Marin Municipal Water District funds through the State Water Resources Control Board.

We thank the following reviewers for their thoughtful assistance:

Leslie Ferguson, San Francisco Bay Regional Water Quality Control Board
Liz Lewis, Marin County Department of Public Works
Mischon Martin, Marin County Open Space District
Ruth Pratt, Marin County Department of Public Works
Nancy Scolari, Marin Resource Conservation District

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Other Illustrations: Lisa Krieshok, Susan Pinkerton
Cover photo by Brock Dolman, www.oaecwater.org

This handbook, in both its first incarnation and this revision, is a collective creation. We are deeply grateful to everyone who contributed—the practitioners who tried and improved many of the techniques, the landowners and agencies who encouraged experimentation, and all who worked to revive *Groundwork.* This revision is dedicated to Don McIsaac, George Flanders and Leo Cronin, who each in his own way, helped keep the coho returning to Lagunitas Creek.

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Baylands Ecosystem

Habitat Goals

A Report of Habitat Recommendations

Prepared by the San Francisco Bay Area Wetlands Ecosystem Goals Project

Please note that news about this Report will be posted periodically on SFEI's website at www.sfei.org.

To order additional copies of this report, please contact:

San Francisco Estuary Project
c/o S.F. Bay Regional Water Quality Control Board
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Oakland, CA 94612
(510) 622 – 2465

Please cite this report as:

Goals Project. 1999. Baylands Ecosystem Habitat Goals. A report of habitat recommendations prepared by the San Francisco Bay Area Wetlands Ecosystem Goals Project. U.S. Environmental Protection Agency, San Francisco, Calif./S.F. Bay Regional Water Quality Control Board, Oakland, Calif.

This project has been funded in part by:

- United States Environmental Protection Agency Assistance Agreement Nos. X-999614-01-0 and CD 999454-01-0 to the California State Water Resources Control Board and by contract No. 6-063-120-0 in the amount of $125,000;
- United States Environmental Protection Agency Assistance Agreement No. X-999782-01-0 to the San Francisco Estuary Institute;
- California Department of Fish and Game contract No. FG-6587/R-3 in the amount of $60,000;
- California Department of Fish and Game contract No. FG-6461NH in the amount of $50,000.

The contents of this document do not necessarily reflect the views and policies of the Environmental Protection Agency, the State Water Resources Control Board, or the State Department of Fish and Game, nor does mention of trade names, commercial products, or private firms constitute their endorsement or recommendation for use.

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Napa County
Integrated Water Resource Management Planning Framework

(Napa County IWRMPF)

REPORT

April 2011

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